Development of technology for the construction of low-cost road embankments

Gift Nwadinma Obuzor

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Certificate of Research

This is to certify that except where specific reference is made, the work described in this thesis is the result of work carried out by the candidate. Neither this thesis, nor any part of it, has been presented or is currently submitted in candidature for any other degree at any other university.

..... 04/11/2011 .....

Gift Nwadinma Obuzor (Date)

(Candidate)

..... 04/11/2011 .....

Dr John M. Kinuthia (Date)

(Director of Studies)

..... 04/11/2011 .....

Dr Rod B Robinson (Date)

(Supervisor)

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Extended abstract

It is envisaged that flood plains will be put into more active usage to meet the increasing demands for road infrastructural development as well as relieve the pressure exerted on arable lands owing to infrastructural development activities. This is consequent upon the general shortfall in the availability of soils possessing the right engineering properties to carry infrastructures such as roads which consume large tracks of land. Expanding the global infrastructural base is inevitable due to the ever increasing human population and the need to meet their social, economic, political and transportation needs. However, owing to the prevailing environmental awareness campaigns fronted by different environmental agencies, there is the need to regulate and monitor the interaction of the processes involved in the provision of these needs with the limited resources as well as the environmental aftermath associated with such operations. The stabilization of flood plain soils for road embankment construction is envisaged to reduce the demand on the material resources required to build classical high embankments in flood prone areas as well as offer implied mitigating dimensions in the restoration of environmental integrity. This impliedly will reduce the use of traditionally unsustainable methods of soil stabilization such as, the excavation and importation of new materials, to a more robust system that will offer environmental friendliness amidst value engineering for better strength and durability results.

The experimental processes involved the simulation of flooding scenarios in the laboratory, to monitor the strength and durability aspects of low-bearing-capacity soils (such as Lower Oxford Clay) stabilized with blended mixes of the traditional stabilizer of lime and the novel materials of lime and Ground Granulated Blastfurnace Slag (GGBS) by-product combined. Preliminary investigations were carried out on the Lower Oxford Clay soil to establish the moisture and compaction requirements of the material. Different mix compositions were formulated by incrementally replacing the amount of lime in the system with GGBS. This was based on the premise that high stabilizer contents could offer better stabilization to flood susceptible geo-materials upon flooding. A high stabilizer level of 16% was therefore investigated. Regimes of different blending ratios were established as follows: 16%Lime-0%GGBS, 12%Lime-4%GGBS, 8%Lime-8%GGBS, 4%Lime-

12%GGBS and 0%Lime-16%GGBS and tested at moisture contents of 23%, 28%, 33% and 38%. The two extremes 16%Lime-0%GGBS and 0%Lime-16GGBS were used as controls.

A system of elimination based on strength criteria was employed, where only the 8%Lime-8%GGBS and 4%Lime-12%GGBS mixtures were deemed fit to be investigated further to determine their resistance to challenging environmental factors of flooding. The test samples were cylindrical, measuring 50 mm in diameter and 100 mm long, and these were compacted using a static compaction apparatus to achieve Maximum Dry Density (MDD). Depending on the testing regime to be applied to a given specimen, a curing pattern was defined and samples were wrapped in cling film to minimise moisture losses. At the end of each curing period of 7, 14, 28, 56 and 90 days, one of the experimental procedures which ranged from Unconfined Compressive Strength, Water Absorption, Volume Stability, Permeability, Soaked Strength and Durability Index Assessment or Compressibility Assessment was carried out on the moist cured samples. Following these assessments, the 4%Lime-12%GGBS mix composition was appraised to have overall improved characteristics with the added benefit of reduced cost of material utilisation.

Based on the available data, regression analyses were carried out and equations established for predicting the strength values of stabilized materials. Using these equations further extrapolations were made and the observable trends were those of the dependence of compressive strength on the age of moist curing and the compaction moisture contents at which samples were produced at given blended mixture.

Cost-benefit-risk analysis was also carried out with a further cost annualisation of the capital and operational cost of a selected system. It is reassuring to learn that at replacement level of lime with GGBS of 4%Lime-12%GGBS it was possible to establish multi-binder mixtures that could be effectively used for sustainable construction in flood prone areas with enormous savings accruing from the possible higher strength and

enhanced durability indices achievable over traditional unsustainable options of continued over-reliance on lime and Portland cement.

List of Abbreviations and Symbols

A	Cross-sectional area
Al_2O_3	Aluminium Oxide
aq	Aqueous
A(t,r)	Present value of annuity factor
Ave.	Average
B_1, B_2, B_3	Boreholes 1, 2 and 3
BS	British Standard
C	Centre
$CaCl_2$	Calcium Chloride
$CaCO_3$	Calcium Carbonate
C-A-H	Calcium Aluminate Hydrate
CaO	Calcium Oxide
$Ca(OH)_2$	Calcium Hydroxide
CASH	Calcium Alumino Silicate Hydrate
Ca^{2+}	Calcium ion
CBR	California Bearing Ratio
CFC	Chlorofluorocarbons
cm^3	cubic centimetre
CO_2	Carbon Dioxide
CSS	Completely Soaked Samples
C-S-H	Calcium Silicate Hydrate
D	Diameter
DC	Dynamic Consolidation
dh	head loss over a flow path
DI	Durability Index

Dia.	Diameter
EAC	Equivalent Annual Cost
e.g	for example
etc	et cetera
Eq.	Equation
Fig.	Figure
g	gram
GGBS	Ground Granulated Blastfurnace Slag
G.L	Ground Level
g/l	Gram per litre
g/mol	gram per mole
H	Horizontal
H_2O	Water
I	hydraulic gradient
ISAT	Initial Surface Absorption Test
Ip	Plasticity index
K	coefficient of permeability
kg	kilogram
kg/m^3	kilo gram per cubic meter
kJ/mol	kilo joules per mole
kN	Kilo Newton
kNm	Kilo Newton Meter
kN/m^2	kilo Newton per square meter
kPa	Kilo Pascal
K_o	at-rest earth pressure coefficient
LADPW	Los Angeles County Department of Public Works

LE	Linear Expansion
LL	Liquid Limit
L_1L_2	Distance to the left
LOC	Lower Oxford Clay
Ltd.	Limited
m	Meter
m/c	Moisture Content
MDD	Maximum Dry Density
Mg/m^3	Mega gram per cubic meter
MINITAB	Multi regression software
mm	Millimetres
$m^3/m^2/day$	cubic meter per square meter per day
m/s	meters per second
N	Newton
$NaCl_2$	Sodium Chloride
nm	nanometer
NPV	Net Present Value
NPW	Net Present Worth
OMC	Optimum Moisture Content
OPC	Ordinary Portland cement
OS	Original Samples
O_3	Ozone
PC	Portland cement
Pfa	Pulverised Fuel Ash
pH	Measure of acidity or alkaline
PI	Plasticity Index

PL	Plastic Limit
PSS	Partially Soaked Samples
PVC	Polyvinylchloride
PVDs	Prefabricated vertical drains
Q	Flow rate
r	rate of loan or capital cost
R_1R_2	Distance to the right
SI	Standard International unit
SiO_2	Silicon Oxide
SRI	Strength Reduction Index
S_1, S_2, S_3, S_4	Sample locations
t	tonnes
t	time of lifespan
TRL	Transport Research Laboratory
UCS	Unconfined Compresive Strength
UK	United Kingdom
UN	United Nations
US	United States
V	Vertical
Vs.	Versus
WHO	World Health Organization
Ws	Weight of Soil
WSA	Wastepaper Sludge Ash
Wst	Weight of Stabilizer
wt.%	Weight percent
Ww	Weight of water

Y	Unit weight of sand
Z	Depth below ground surface
δ	sand-micropile friction angle
δ_h	lateral earth pressure
%	Percentage
^oC	Degree Celsus
μm	micrometer
ω_L	Liquid Limit
ω_p	Plastic limit
ρ_d	dry density
v	Apparent velocity of flow
4L-12G	4%Lime-12%GGBS
8L-8G	8%Lime-8%GGBS
16L-0G	16%Lime-0%GGBS
ΔH	Temperature change

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1.0 Chapter One: Introduction

1.1 Road and Economic Development

Transport as a generic term referring to the movement of persons, goods and information in space has become one of the key economic activities in modern societies (Cundrič et al, 2008). In most of the developing countries road transport is the most dominant mode of inland transportation. In Ghana, it has been estimated that over 90% of the national freight and passenger traffic inland, rely on road transport (Frempong, 1999). The importance of road infrastructural development cannot be overemphasized, especially to the rural poor who depend largely on road as their main channel of communication in carrying out their daily routine chores. If poverty alleviation is to be achieved, then, greater interest should be invested in road infrastructural development. The need for road infrastructure development could be as insignificant as providing and improving mobility requirements between settlements (Richards 1984; Barrios, 2008). Other important functions of rural roads according to Richards (1984) and Barrios (2008) include lowering transportation costs, lowering time spent on transportation, bringing access to mechanised transport closer to the home and for longer period of the year, expanding domestic food production, and stimulating export of agricultural produce amongst others. These benefits stimulate trade and migration bringing about economic improvement (Richards 1984).

Transportation should be a driver of, and not a response to, economic development. The absence of road infrastructural development has been seen to slow down economic growth. Good transportation links for a country is therefore a pre-requisite for economic growth and regeneration in manufacturing, agriculture, commerce and tourism for example, requires the need to deliver on a day to day basis. As Howe (1984) puts it, 'it is a significant factor in the development of industry, in the expansion of trade, in the conduct of health and education programmes, and in the exchange of ideas'. The ability to move materials is an integral part of all sectors (Howe, 1984). Wealth, according to Howe (1984), is highly correlated with mobility and poverty with immobility in terms of access both to roads and vehicles. Therefore, for efficiency in business and politics, transport infrastructure is important and it also reinforces the confidence of investors which has an implication of economic growth. According to Andersson and Andersson (2008),

economic development always takes place on an arena of hard and soft infrastructures. The soft infrastructure is made up of generally accessible knowledge (e.g., a shared spoken language and writing system, technological know-how, and organization practices), political and legal systems, and informal rules of behaviour. The hard, physical, infrastructure mainly consists of the built environment, and transport and communication networks, which link producers and consumers in different locations to one another. Infrastructure investments (road) have been proposed in the development literature as an influential factor of economic growth (Fedderke and Bogetić, 2009; Fedderke et al, 2006; Cundrič et al, 2008; Olsson J., 2009; Botham R. W., 1980; Barrios, 2008). Fedderke and Bogetić (2009) hypothesized that infrastructure investment (road) facilitate private investments by lowering production costs and opening new markets, thereby creating new production, trade and profit opportunities. Olsson (2009) investigated the effect of improved road accessibility in some parts of Philippines by assessing the creation and response to economic opportunities of individuals and companies in the study area. It was concluded that while road project was not the sole cause of the social and economic changes identified, it did facilitate trade and investment, increased competition, and released a previously under-utilized production potential. It also brought about efficiency gains in terms of time savings which is particularly in favour of an agro-based economy (significant in the case of perishable goods). However, there is a potential conflict between business concerns for increasing the size of the road network, and environmental interests which have an opposing view. Indeed, the pollution and environmental consequences of new road construction, or enhanced use, at particular locations can act as a disincentive to economic activity, such as tourism.

Road construction projects are often faced with the challenge of high cost of construction. According to Frempong and Tsidzi (1999), road construction costs can be very high because of the limited opportunity of locating suitable base materials within economic haul distances. The author argued that since earthworks constitute a significant percentage of road construction costs, any form of construction which will reduce the cost of the earthworks component without introducing additional maintenance or vehicle operating costs should be a welcome relief to the construction community and low-income economies.

The issue of earthwork cost component in road construction is more critical to flood plains where high embankments far above the flood life-line are required for road construction. Secondly, flood plains do not possess the geological competence to produce high quality engineering geo-materials for road sub-base and base construction due to their depositional characteristics. Hence, excavation and importation of materials from other sites are employed. This has both cost and environmental considerations that are becoming increasingly unbearable. The current work addresses the technological and environmental issues related with solving this problem. It is envisage therefore that the current work will make inroads towards the provision of the required solution by enhancing the strength and durability requirements of low capacity soils upon flooding. The actualization of this feat will not only represent cost saving accruing from reduced road embankment heights to within the flood life-line but will also mean enormous savings from reduced frequency of maintenance and re-construction work on flood plain roads and also contribute to global energy reduction and materials conservation campaigns. The cost of transporting borrowed materials will be eliminated. This has both cost and environmental implications of the reduction in fossil fuel utilized and carbon footprint.

1.2 Effects of Climate Change Issues

The discharge of industrial effluents using deep sea clarifiers and dumping of waste in landfill dump sites was well favoured until recently. Bans and high taxes have now been placed on such practices, making the choice of either waste recycling or management an unambiguous option. With this increasing environmental awareness, certain industrial operations have been identified to be associated with the production of gases and chemical compounds with detrimental global environmental effects. Predominantly, the compounds identified are; water vapour, carbon dioxide CO_2, ozone O_3, methane, nitrous oxides and chlorofluorocarbons CFC (Tzu-ping Lin, 2009; Ensor and Berger, 2009). These have been appraised to posses the attribute of absorbing the energy radiated back into the atmosphere from the solar system. This energy absorption process triggers an increase in global temperature profile, thereby, causing an alarming increase in the temperature of the earth surface. Research has also identified this phenomenon known as the greenhouse effect to be responsible for climate change and global warming. According to Ensor and Berger

(2009) Climate change first came on to the international political agenda in 1992 at the UN Conference on Environment and Development in Rio de Janeiro where it was discussed as an environmental problem. The practical implications of the climate change include raising sea levels as a consequence of melting icebergs in the oceans, and increased rainfall occurrence, severity and duration (CIRIA, 2004; Ensor and Berger, 2009) resulting from abridged water cycle. The cyclic movement of water from one phase to the other will likely be affected as temperature rises, with the consequence of increasing transpiration and evaporation, all this culminates in flooding. Ensor and Berger (2009) pointed out that climate change presents multiple dangers – river flooding and riverbank erosion, which are becoming more severe as rainfall intensity and glacial run-off rates change, while in some flatlands rising sea level threatens millions of people with flooding and long term salinization. Ensor and Berger (2009) also identified that the impact of climate change could differ with altitude, in their words, 'in the mountains, global temperature rise is leading to glacial retreat, with increased risk of glacial lake outburst floods. Increased run-off variability and sediment loading from glacial melting have an adverse effect on hydropower generation, irrigation and rural livelihoods'. These also incubate an applied consequence of over reliance on fossil fuel. The inevitability of flooding owing to changing global environmental conditions could be expressly read from the words of the Environment Minister of the Welsh Assembly Government, Jane Davidson, in her words: "if we are to adapt successfully to climate change, then the way we manage our shores and rivers will be critical" (BBC NEWS). Waltham (2009) envisages a rise in the height of flood peaks as a consequence of changes in land use - urbanization and deforestation, with an unprecedented effect on floodplain hydrology.

The manufacture of construction materials such as Portland cement and lime presents great environmental hazards. These hazards could be viewed from two different perspectives; (1.) the depletion of natural raw materials (Lilley, 1975), and (2.) energy consumption (Higgins, 2005). It has been proven that the production of Portland cement which involves the heating of limestone and clay in an inclined rotary kiln to temperatures well above 1350°C releases carbon dioxide. In practical terms, one and a half tonnes of raw materials are required to produce one tonne of Portland cement, giving out one tonne of carbon dioxide emission as waste to the environment. Driven by sustainability objectives, attempts

have been made to replace cement in concrete with industrial waste materials such as Pulverised Fuel Ash (pfa), Ground Granulated Blastfurnace Slag (GGBS) (Lilley, 1975) and/or recycled materials in the form of powder from recycled brick works. All these endeavours are aimed at reducing the CO_2 emissions per tonne of concrete utilised, with the intention of reducing the greenhouse effect and its remote consequences, and subsequently reducing raw material usage. As a way forward, these industrial activities as well as the application areas of cement cannot be completely eliminated. The activities can however be reduced by accepting sustainability, hence, the pressure exerted on land will obviously linger as new development interests cannot be left out (Lilley, 1975). The far reaching effect of this is the unavailability of soils possessing the required engineering profile to sustain future developments particularly road construction (Lilley, 1975; Tomlinson, 1975). This predicament most often drives the innovation enjoyed by the civil engineering community, with soil stabilization options offering some of the possible design solutions (Lilley, 1975).

It is envisaged that flood plains are going to be actively put into use by future development efforts. This is because of the shortage in good engineering soil resulting from the cumulative impact of industrialization on land over the years. However, these flood plains lack the geological competence to produce soils with the right engineering properties to be used as road sub-base and sub-grade materials without further improvement. For lasting road pavements to be successfully built, the benefits of several soil improvement options have to be weighed against each other (Lilley, 1975). However, there is no hard and fast rule to stabilization of troublesome or low bearing capacity soils. The only guiding principle is cost and applicability of the proffered solution. Over the years, attempts have been made to improve soils with different short falls using different solutions. Predominantly, lime has been used for improving clay soils, but where there is the presence of sulphates, there is usually the apprehension of possible swelling (Puppala et al, 2005; Wild et al, 1999). Obviously, high embankments are used when building roads in flood-prone areas. This is to ensure that the road structural layers are above the reach of flood waters. However, during flooding events these high embankments are highly distressed, washed off or subjected to cracking due to freeze-thaw action on the soil. This has an implication of increasing the whole life costing of road infrastructures built on flood

plains. The current research aims to find solutions to this problem of engineering flood plain soils for sustainable road construction. It is envisaged that the use of marginal industrial waste materials will not only yield the benefit of improved strength for local geo-materials in flood plains but also reduce void spaces and thus altering the permeability characteristics. To achieve this aim, Lower Oxford Clay soil was investigated by the addition of Lime blended stabilizers. This stems from the background knowledge of the difficulty of engineering this soil type for civil engineering applications using lime on its own. It is believed that if the problems posed by this soil are manageable, then, any other scenario presented by flood plain soil could to a large extent be manageable. This has the implication of enormous savings from the reduction of embankment heights and improved durability of roads constructed on flood plains.

1.3 Cost of Construction

The cost of earthworks and drainage construction in road works represent a huge chunk of the total cost of constructing roads - up to 40% of the total cost. The factoring of this earthworks component into the overall cost of road projects could even be more where high embankments are required such as flood plains, or where weak soil is encountered thereby requiring long distant haulage for the transportation of good construction materials. This practice not only adds to the overall cost of construction but also has the implication of increasing carbon footage amidst salient contributions to global natural resource depletion. For details of cost reduction refer to Section 6.2.7.

1.4 Aims and Objectives

In view of the developmental, technological and environmental issues discussed so far, the current research has tailored its aims and objectives to demonstrate the following:

1. That even the poorest quality soil can be improved to a degree that infrastructure may to be built on them.
2. That environmental conditions leading to flooding are surmountable where significant interest is invested in development.
3. That cost effectiveness, durability and strength criteria for constructions on poor quality soils can be successfully improved by the incorporation of industrial waste materials bearing cementitious qualities.

4. That reusing, recycling and management of resources can reduce the emission of carbon dioxide into the atmosphere, and the possible danger of resource depletion.

Pursuing the above objectives resulted in the pursuance of the following other objectives:

1. The reduction of the cost of road transport by effective provision and management of sustainable road infrastructural network development, particularly, in areas prone to flood inundation, where road infrastructure suffers loss of strength and premature failure due to water ingress into road structural layers.
2. Improvement of the mobility of rural and urban poor for meeting their livelihood needs, ensuing from lowering transport cost and lowering time spent on transportation with the implication of expanding domestic food production, and stimulating export of agricultural produce.

1.5 Contribution to Knowledge

Driven by these aims and objectives, a body of work was carried out in order to evaluate existing knowledge on the subject matter, but also to contribute to this existing knowledge as illustrated by the following:

1. Demonstration that the stabilization of flood susceptible clay soils for road embankment construction can be achieved at a high stabilizer addition of up to 16%.
2. Illustrating that Lime when blended with GGBS achieves its utmost strength when used only in small quantities relative to the industrial by-product, especially when combined at certain carefully established workable ratios dependent on the type of soil and moisture content.
3. Showing that certain Lime-GGBS stabilizer blends are very good at mitigating the swelling potential associated with the stabilization of sulphate-bearing clay soils with lime. This is thought to have been made possible by GGBS reducing the free lime in the system, as well as enhancing the formation of vital cementing gels.
4. Establishing that Lime-GGBS blended stabilizer has been established to develop strength even when soaked in water within the critical early stages of strength formation. They are also noted to increase permeability and reduce water absorption.

1.6 Expected Beneficiaries

1. Providers of the transportation network – lower maintenance cost and the need for less frequent major reconstruction of roads.
2. The travelling public (particularly the low-income people who make greatest use of public transport) – improved and reliable communication networks (less time lost in travelling), improved distribution of supplies at lower cost.
3. Haulage and transport organisations – lower vehicle maintenance costs, enhanced working life of vehicles, less fuel used (thereby producing less pollution and reducing the price of distributed goods), able to plan operations and be efficient.
4. Local workforce – additional work opportunities in manufacturing and construction industries, easier movement to place of work, better transport of their produce to market, skills enhancement.
5. The community – improved efficiency of usage of renewable resources, waste reduction, development of sustainable industry, conservation of monetary resources, protection of the environment.
6. The research community – contribution to the pursuance of sustainability, by way of utilising natural, industrial and agricultural waste and by-product materials as a contribution to the solution to the global challenge of attempting to mitigate the effects of climate change.

1.7 Structure of Thesis

The research presented in this thesis is a laboratory assessment of the different parameters that undermine strength and durability criteria of low-bearing capacity and expansive clay soils, particularly when and where they are subjected to unfavourable environmental conditions of flooding. The work is organised into seven chapters, the current chapter forms an introduction, and aims to lay the basis for the research work, by justifying the need to embark on this project and subsequently, aid the reader to navigate effortlessly through the whole thesis. The chapter also highlights, at this early stage, the findings of this work as contribution to knowledge. Chapter Two is a review of the current literature; this is an appraisal of the development of knowledge to ascertain the gap areas in the

understanding of treating troublesome soils especially where flooding is a major concern. This review necessitated the state-of-the-art solution proposed by this research, after a holistic weighting of the merits and demerits of all the available options. Chapter Three gives the justification, sources and properties (chemical and physical properties) of the different materials used in the research, viz: Portland cement (PC), Lime (L), Ground Granulated Blastfurnace Slag (GGBS), Lower Oxford Clay (LOC) and water. With this background, the understanding of the possible reactions mechanisms involving the various materials is made relatively easier. The research methodology is the theme for Chapter Four, whereby, all the experimental procedures are exhaustively reported in detail. Coloured Plates were employed to enhance the clarity and understanding of the various systems. In Chapter Five, the results of the different experiments carried out are presented. The first section presents the basic material characterization tests conducted on the soil. Then the dry material properties were investigated. This started with the Unconfined Compressive Strength (UCS), Volume Stability and material Compressibility. These parameters were measured against those of the control or baseline experimental values. This was followed by the section dealing with material hydric properties (durability criteria) which investigated the Water Absorption and permeability, and their influence on other material properties like Soaked Strength, Strength Reduction Index and Volume Stability. Finally, the compressibility upon loading was also determined and reported in the last section of chapter five. Chapter Six discusses the results, observing the different trends and assessing the possible outcomes against baseline standards and established knowledge. After this detailed discussion, which also includes mechanisms and various hypotheses it was prudent to briefly point out the practical implications of the entire research work. This is in the end the impact of the entire work in terms of civil engineering practice. In Chapter Seven, the research findings are summarised and conclusions drawn. Areas for further research interest are also highlighted.

Chapter Two: Literature Review

2.1 Flood Plains

Flood plains are low-lying areas adjacent to water bodies (oceans, rivers, lakes, ponds e.t.c) which stand the risk of seasonal inundation (Bridge J. S., 2003; CIRIA, 2004). Waltham (2009) defined a floodplain as the zone of alluvial deposition along valley floor, subject to periodic flooding. The alluvium builds up over time, much of it formed as over-bank deposits, which are mostly fine grained and horizontally bedded. Alluvium according to Waltham (2009) is characterized by laterally and vertically variable materials ranging from clay to boulders and possessing a wide range of engineering properties. According to Bridge (2003) floodplains may be well defined or interwoven especially in deltaic regions with scattered rivers, streams, tributaries and estuaries. RSPB, EN, and ITE (1997) distinguished six levels of flood plains in the UK: semi-natural floodplain, washland, water meadows, ponds and lakeside lowlands, drainage channels and intensive water level management low land areas. These are mostly artificial floodplains serving as flood mitigating measures as most rivers are engineered and regulated to reduce naturally functioning floodplains (RSPB, EN and ITE, 1997). The engineering of river channels in the developed countries opposes the trend in developing countries with well defined floodplains either due to lack of interest in interfering with natural courses or the lack of technical know-how to engage in this form of advanced engineering practices. Floodplains could be active or passive and this depends on the human activities in the floodplain targeted at alleviating the effect of flooding events according to CIRIA (2004). A flood plain could be termed active when it experiences regular periodic flooding, whereas in the contrary it becomes a passive flood plain. Flood defence works are particular responsible for this distinction as they shut out flood plains from regular periodic inundation.

2.2 Geological formation of Flood Plains

According to Bridge (2003) floodplains are underlain by the deposits of river channels, of over-bank floods, and of lakes. Floodplain widths could vary from channel-belt to many tens of channel-belt widths. The geometry of floodplains changes with time as a result of: channel migration within the channel-belt; migration, cutting, and filling of floodplain channels; large-scale movements of channel belts; local tectonism; progressive deposition

or erosion (Bridge, 2003). The geological formations of floodplains are dependent on the water flow patterns and sediment transportation characteristics of a given channel which are functions of the channels width, topography and the varying quantity and size of available sediment (Bridge, 2003). The author, however, identified that the patterns of flow and sediment transport on natural floodplains are difficult to ascertain owing to the difficulties associated with observations during over-bank flooding events. Sediment is transported over the floodplain as bed load and suspended load during over-bank floods, and these sediments are usually in the form of mud pellets, soil concretions, and organic debris, deposited in basic sedimentation units of millimetre-thick to decimetre-thick strata sets over a long period of time (Bridge, 2003). The strata sets according to the author may be sheet-like, wedge-shaped, or lenticular, depending on the local environment of the deposition. These depositional characteristics explain why floodplain geo-materials needs engineering, to be effectively used as road sub-base and base materials for sustainable and whole-life-cycle assessment focused road construction. According to Waltham (2009) a soil is classed as a weak soil when it possesses a UCS value of less than 600 kPa. Cohesive clay alluvium deposits on floodplains possess strength values in the range of 0-200 kPa depending largely on the consolidation history of the soil.

2.3 Predominant Soil Types in Flood Plains

Provided there are no human influences to reduce the effect of such factors as deposition rate, parent materials, groundwater composition, climate, and vegetation, the resultant effect over many years will be soil formation. According to Bridge (2003) floodplains are characterised by the prevalence of varying quantities of fine to very-fine sand, silt, and clay and depending on stage in earth history and climatic zone, could also contain abundant plant debris, shells of freshwater molluscs, vertebrate bones etc.

2.4 Different Soil Improvement Options

Soil improvement techniques abound, and there is no hard and fast rule to soil stabilization (Chu et al, 2006; Ayadat et al, 2008; Nusier et al, 2007). Currently, there is little or no literature on the stabilization of flood susceptible soils. However, available literature on the general area of soil stabilization will suffice as guidance to the rigours of soil stabilization

reactions. According to a report by Little et al (2000), stabilization projects are almost always site-specific, requiring the application of standard test methods, along with fundamental analysis and design procedures, to develop an acceptable solution. Little et al (2000) and Degirmenci et al (2007) reported that the principal materials used for the cementitious and modifications of soils and highway pavement materials are lime, fly ash and Portland cement, with fly ash as a by-product of other industrial activities. Little et al (2000) further asserted that the stabilization of soils and pavement bases with fly ash is gaining increasing popularity amongst design engineers. Wild et al (1998) driven by global environmental changes conducted a series of researches aimed at reducing the percentage of lime utilised in soil stabilization projects and concluded that, partial substitution of lime with GGBS gave an improved 7 and 28 days strength for different classes of clay soil. Efforts were also exerted by Alshawabkeh and Sheahan (2003) in assessing the possibility of improving soft soil by ionic injection under electric fields and their findings proved that ionic injection is a viable soil treatment option especially where much disturbance to the soil is an issue and the soil has low hydraulic conductivity. Attom and Al-Sharif (1998) in a desperate bid to reduce the prevalence of olive wastes in Jordan, investigated the use of burnt olive waste in the stabilization of weak and expansive soils. Similarly, the drive to reduce the increasing prevalence of industrial waste triggered the investigation by Kamon and Nontananandh (1991) to evaluate the potential for burning various industrial wastes, of predetermined proportions, to give a by-product that has self-cementing characteristics similar to those of OPC and used for soil stabilization. Amongst the reasons for stabilizing a soil, the cost aspect cannot be overlooked (Phanikumar et al, 2008). As Lee and Karunaratne (2007) puts it, "a considerable part of the cost incurred in the construction of highway embankments on soft ground arises from the need to pre-treat the soft soil in the foundation. If proper soil treatment is not carried out, protracted and costly road maintenance cannot be avoided". The improvement techniques of troublesome soils could be disaggregated to various key headings for clarity.

2.4.1 Physical Approach

A wide range of physical practices are used to improve the carrying capacity of weak soils. These include excavation and replacement of soft (clay) soils with selected fill materials; minimizing moisture ingress to the soil; the use of compaction equipment; alteration of soil

gradation or mechanical means etc (Prusinski and Bhattacharja, 1999). The problems associated with this type of soil improvement option include the cost of disposal of significant quantities of inferior materials or the use and transportation of additional virgin materials (Prusinski and Bhattacharja, 1999), which negates the efforts aimed at the millennium agenda, of reducing carbon footprints and also reducing natural resource depletion.

2.4.1.1 Excavation/Importation

Problems associated with expansive soils in sub-grades complicate construction activities around the world according to a report by Yang et al (2007), where as part of efforts to construct a link road between China and other countries in southern Asia, roughly two million cubic meters of expansive soil was excavated. Replacing this large volume of excavated soil with virgin soil is uneconomical and harmful to the environment (Yang et al, 2007; Prusinski and Bhattacharja, 1999). In contrast, Arulrajah et al (2009) reported that this could be economical if the soft soil is only in little quantity.

2.4.1.2 Mechanical or Granular Stabilization

According to O'Flaherty (2002) mechanical or granular stabilization of soil is the process whereby the gradation of raw soil is improved by blending a coarse and/or fine material, with the aim of achieving a dense homogeneous mass when compacted. In general, mechanical stabilisation is effective when the sand and clay contents are 60-90% and 10-30% respectively, the remainder being the silt fraction. This is true for most soils with the exception of laterites. Furthermore, O'Flaherty (2002) recommended the addition of chlorides ($NaCl_2$ or $CaCl_2$) as a way of enhancing the stability of mechanical or granular stabilised soil, as these salts have excellent hygroscopic and deliquescent properties which are very useful for unsealed roads. Hygroscopic property is the ability of a substance to attract water molecules from the surrounding environment through either absorption or adsorption. With unsealed roads the high level of dust produced during traffic operations could be alarming, hazardous, and constitute environment pollution. The hygroscopic and deliquescent properties help in reducing the formation of dust. However, most substances dissolve in the absorbed water which is a property known as deliquescence. Bringing it to

context, unsealed roads present environmental danger in dry season due to dust and are also liable to subsidence when moisture is very low. These problems are minimized by the addition of materials with excellent hygroscopic and deliquescent properties which tend to retain moisture absorbed from the environment.

2.4.1.3 Compaction

One of the easiest ways of improving the strength of weak soil is by – static or vibratory compaction, although many commentators do not consider it as a form of stabilising soil, either due to its simplicity or its limitation to granular soil types. To give compaction a wider recognition, researchers like Ausilio and Conte (2007) and Arulrajah et al (2009) reported the stabilisation of soft soil materials by the installation of stone columns otherwise known as the vibro-replacement technique (refer to Section 2.4.2.4.7 for details). The underlying principle involved the installation of large-sized columns of well-compacted, coarse-grained backfill materials. These columns were found to increase the overall stiffness and shear strength of the soft soil, thus permitting the adoption of greater design loading. Another state-of-the-art innovation in soil compaction practice is the dynamic consolidation method of soil compaction (see Fig. 2.2).

2.4.1.3.1 Dynamic Consolidation method

This method was originally proposed by Menard and Bruise (1975) for fine-grained soils but recent attempts made by applying Dynamic Consolidation (DC) to treat soft grounds recorded a huge success. The principle underlying this method is the typical use of a flat cone-shaped steel hammer with top and bottom diameters of 2.0 and 2.5 meters respectively, and having a self-weight of 120kN. This is suspended to a given height above ground level with the use of a hoist or crane, which raises and releases the hammer to hit the ground repeatedly in a controlled manner for a number of blows depending on the degree and depth of compaction desired.

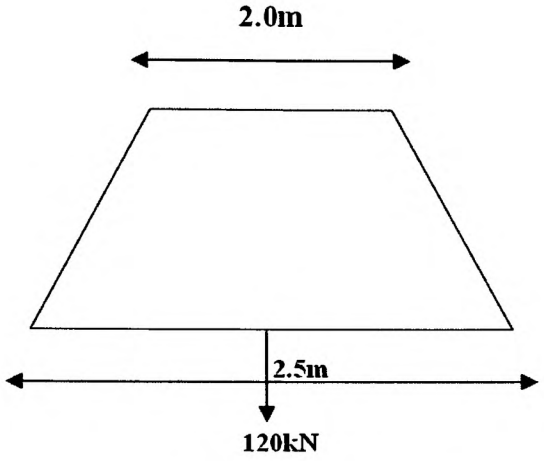


Fig. 2.1. A Typical flat cone-shaped hammer

There are basically three levels of compaction energies associated with this technique: 520kNm (small); 810kNm (medium) and the 1750kNm (high) compactive energies. In practice, the 520kNm (small) should be used to start the DC process as this will enhance the consolidation of the top soil, after which the medium level energy should be used as it provides a deeper compaction which will accommodate a number of projects. The use of the 1750kNm compaction energy is usually applied with caution to avoid the destruction of the micro structure of the target soil to be improved (Chu et al 2006).

Field experiments have shown that it is better to install a proper drainage system to dissipate the pore water pressure as this will not lead to any increase in shear strength. However, for greater effectiveness, this should be employed with another technique possibly the use of pre-fabricated vertical drains (PVDs). A sand blanket is preferable as this provides an effective drainage network (Chu et al 2006; Arslan et al, 2007).

Fig. 2.2. Dynamic consolidation in operation (After Chu et al 2006).

2.4.2 Chemical Approach/Stabilisation

The choice of any stabilization design solution depends largely on the limiting borders set by economic constraints, time availability, accessibility to site, soil types and mineralogical compositions, as well as the prevailing environmental conditions. In most cases, the equipment needed to carry out chemical stabilizations could be as little as hand augers to effect the boring of holes for the injection of suitable chemical substances. Lime and Portland cement are the two primary materials in the arena of soil stabilization using chemical additives. However, innovative thinking has led to the use of other chemicals to perform soil stabilization practices, under pinning the practicality of these options on the knowledge that altering the chemical environment around the soil particles upon which the behaviour of the soil largely depends will produce good results in achieving soil stability (Little et al, 2000). Little et al (2000) evaluated different soil stabilization options and concluded that the use of cementitious materials makes a positive contribution to economic and resource sustainability because it allows enhancement of both standard and substandard in-situ soil to levels consistent with the requirements of a given application. Hughes and Glendinning (2004) showed the site specificity of soil stabilization in their report of the stabilization of the Channel Tunnel Rail Link in the South-East of the UK, using the system of deep dry mix ground improvement of soft peaty clay using blast furnace slag and red gypsum. This option which originated from Scandinavia and Japan where very soft soils of high water contents are reportedly prevalent, involves the mixing of the stabilizer at the depth of the soft soil material using contemporary rotary drilling equipment. In dispensing most chemical stabilization design solutions, the injection method of dispensing chemical is always favoured as cost of execution is usually lower compared to other options. However, the success of this option is dependent on the hydraulic conductivity of the soil in question.

2.4.2.1 Lime

The use of lime as a soil stabilisation material is well established (Little et al, 2000; Al-Rawas et al, 2005; Murty and Krishna, 2006). Particularly in road construction where they are used for modifying sub grade, sub-base, and base soil materials (Bell, 1996). This practice has brought about the realisation of several engineering benefits such as improved

strength; improved resistance to fracture, fatigue, and permanent deformation; improved resilient properties; reduced swelling; and resistance to the damaging effects of moisture. The most substantial improvements in engineering properties of engineered soil are seen in moderately to highly plastic soils, such as heavy clays (Little et al, 2000; Bell, 1996). According to Little et al (2000) the sustained and relatively slow pozzolanic reaction between lime and soil silica and soil alumina, released in the high-PH environment, is key to the effective and durable stabilization in lime-soil mixtures. Lime is used on its own to stabilise soils or in combination with other materials to improve different criteria. It could also be mellowed, whereby time is allowed (24 to 72 hours) between initial application of lime, remixing and compaction. When lime is applied to high plasticity soils, there is usually improvement in the UCS value and this has been establihed to improve with age and the percentage addition of lime. On application of lime to clay soils, the affinity of clay minerals for lime is the prime influence. Until this affinity is satisfied ions are not available for any pozzolanic reactions. Because this lime is fixed in the soil and is not available for other reactions, the process has been referred to as lime fixation. Hence, in the process of lime-soil stabilization it is important to establish the lime demand of the soil. However, for lime soil improvement 1-3 wt.% lime is usually used for modifying the soil properties otherwise known as the lime fixation point, and between 2 and 8 wt.% required for lime stabilization (Bell, 1996).

2.4.2.1.1 Lime Stabilized Soil Columns

These are boreholes of about 500mm in diameter, filled with lime-stabilized soil, installed in places by means of a special auger that bores the holes, injects the binder, mixes it with the soil and thereafter compact the mixture. When the auger is finally withdrawn, it leaves the completed column in place in the ground. The binder is usually lime, giving rise to the name, but in some cases, contrary to using lime alone, a mixture of lime and cement could be used if required. For this installation the colmix process is usually employed (Bachy Limited, 1996) which allows the use of lime or lime plus cement binders owing to cost of mobilizing specialised and relatively heavy, equipment to site. In this case, access to work site could be a major limitation (Brookes et al, 1997).

2.4.2.1.2 Lime Piles

These are boreholes of 200mm diameter filled with quicklime. The process is simple as the installation involves the use of any basic drilling rig and consequently pouring quicklime under pressure to fill it up and compact using wooden stake (see Fig. 2.3) (Glendinning and Rogers, 1996). Owing to the simplicity of equipment used, lime piles could be applied in a site of relatively short lengths on earthworks slope requiring treatment or where access to site is difficult. The only set back is that spoils arise for disposal and this could be an environmental concern (Brookes et al, 1997).

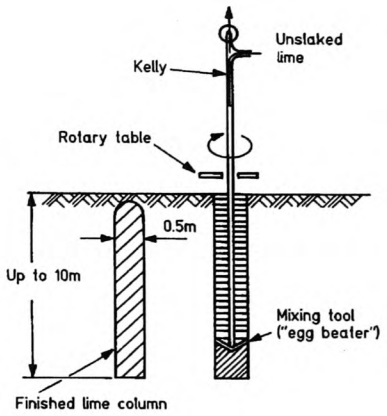


Fig. 2.3 Proceedure for construction of lime columns (after Rogers and Glendinning, 1997)

2.4.2.2 Portland Cement

Studies of cement-treated soil have been conducted for several years (Modmoltin and Voottipruex, 2009; Prusinski and Bhattacharja, 1999). According to Miles (1974) the idea of using a form of cement to induce adhesion between building components was first met at a relatively advanced stage of civilisation. Miles (1974) proposed that earlier structures were composed of earth, sometimes raised in the form of walls or domes by ramming successive layers of earth, or stone blocks, set one above another without the aid of any cementing material. The stability of walls of the latter kind is derived entirely from the regular placing of heavy masses of stone without any assistance from adhesion (Miles D, 1974). The first use of a true mortar according to Miles (1974) dates back to Egypt, where sand and cementitious materials (gypsum) were used to unite blocks and slabs of stone. This gave way to the use of lime by the Greeks, then to the discovery of pozzolanas by the Romans. The Portland cement was invented following the industrial revolution which

greatly stimulated the demand for development. Portland cement consists principally of four compounds which are formed (at temperatures of 1300°C– 1500°C) between lime on the one hand and silica, alumina and iron on the other (Howe and Mamman, 1974). Cement has been extensively used either as soil cement for soil modification or as component for cement stabilization of soil known as cement-treatment. In road construction, the use of cement has been handy in the strengthening of road structural components.

It has been less than two years since the Los Angeles County Department of Public Works (LADPW) officially approved soil-cement use for bank protection and less than six years since the first application was constructed in the County. The soil cement has already been subject to a major durability test. The 2004-2005 winter rain season is the second wettest on record for Los Angeles with 38.18 inches. In the City of Santa Clarita's Newhall area, where most of the soil-cement bank protection has been applied in Los Angeles County, nearly 51 inches has fallen this season. While many unstabilized areas, as well as areas protected by bank protection methods suffered damage, soil-cement in these areas has functioned faultlessly *(Krebs, 2010)*.

2.4.2.3 Chemical Grouting

Different chemicals are used in the processes of stabilising soils, and these might be dispensed to the soil either by injection, boreholes or pond application. For stabilization by injection refer to section on chemical approach/stabilization (Section 2.4.2).

2.4.2.3.1 Stabilization of expansive clay bed using calcium chloride solution

The cyclic volumetric changes experienced by expansive clays upon moisture fluctuations is the product of the intrinsic minerals contained in clays according to a report by Murty and Krishna, (2006). Taking advantage of the sparingly soluble nature of lime in water (about 1.2g/l at 21°C), chemical stabilization of soil using multivalent cations is employed which alters the chemical environment around the clay particles upon which the behaviour of clay depends to a great extent (Murty and Krishna, 2006). These multivalent cations are usually from strong electrolytes such as potassium, magnesium, zinc, ferric, calcium

chlorides and sodium hydroxide. Strong electrolyte being readily soluble in water, could supply adequate cations for exchange reactions is the principle on which this soil stabilization option is based (Murty and Krishna, 2006).

Two processes are involved:

(1) Borehole method: - this method involves the drilling of three boreholes of 100mm diameter and 1.50m deep using hand augers in a triangular pattern at a spacing of 2m centre to centre as illustrated by Fig.2.4, and a predetermined quantity of commercial grade $CaCl_2$ solution filled in the boreholes to saturation point and allowed to diffuse laterally in summer. The quantities of the $CaCl_2$ solution being estimated based on the porosity of the soil. The solution is expected to permeate through the porous materials with the calcium ions going into reaction with the clay mineral components. Favoured by the relatively high solubility of $CaCl_2$ in water the Ca^{2+} engages in cation exchange reaction thereby changing the chemical morphology of the clay-$CaCl_2$ material, the resultant effect being the improvement of liquid limit, plastic limit and swell potentials of the clay. The degree of stabilization is then checked after 3-4 months by collecting sample from positions in-between the boreholes. The much anticipated set back of this option is that there is no cementitious reaction involved to form a matrix structure that will maintain strength development during environmental extremes of erosion or the leaching away of the $CaCl_2$ solution in heavy rain.

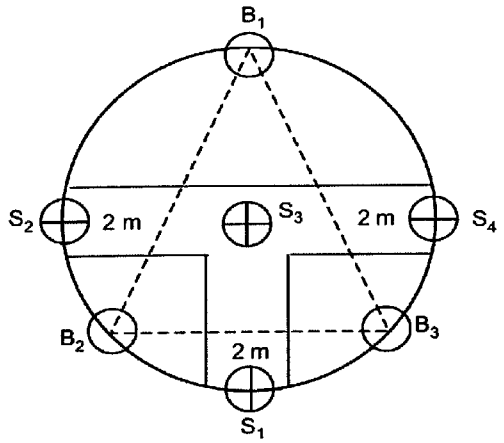


Fig. 2.4 Application of Calcium chloride solution through boreholes – B_1, B_2, B_3: 100mm dia.

(Boreholes filled with $CaCl_2$ solution and S_1, S_2, S_3, S_4: location of samples (After Murty and Krishna, 2006)).

(2) Ponding Method: - this is similar to the borehole method in principle, as discussed above. However, in application, this involves the digging of trenches of 1.2m x 3m x0.3m and a predetermined quantity of commercial-grade $CaCl_2$ solution poured in to saturate the soil bed up to 1.0m depth, estimated based on the porosity of the soil. Heaving is then checked by placing a 450mm x 450mm sized cement concrete reference panel in the centre of the trench and measurements taken as illustrated in Fig. 2.5. The initial reduced level of this is then taken and subsequently, levels are taken at given time intervals.

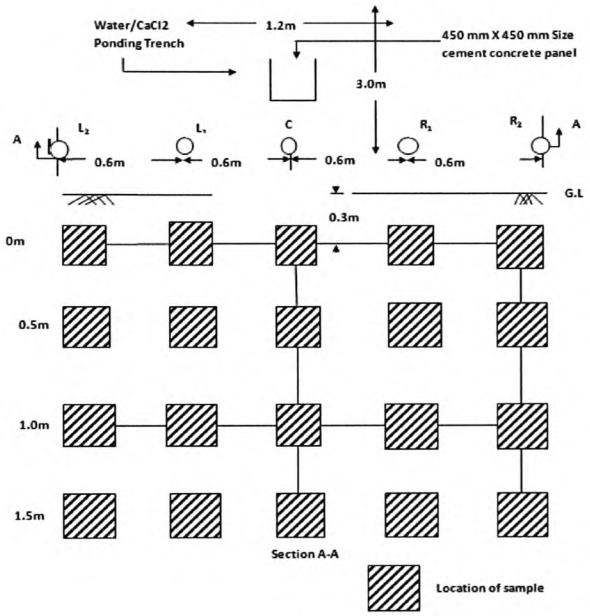


Fig. 2.5 Application of $CaCl_2$ solution by ponding

(After Murty and Krishna, 2006).

2.4.2.4 Other Innovative Options

The prevalence of soils which do not possess the right engineering properties has driven the exploration of different soil improvement options as outlined below. In all the various methods, the cost and environmental tolerance criteria have been seen to be the focal points of interest. Repeatability is not always a key issue as soil improvement projects are almost always site specific in nature especially where cost is a major concern.

2.4.2.4.1 Vacuum Preloading Technique

This method of soil improvement which entails placing a 0.3m thick sand blanket on the surface area to be improved, which is typically a section of 5,000 to 10,000 square metres of soft clay or muddy soil, was first suggested by researchers at the Swedish Geotechnical Institute (Kjellman, 1952). The sand is essentially to enhance consolidation (acting as a surcharge) and to provide a stable working platform before substantial improvement of the soil can take place.

Prefabricated vertical drains, (PVDs) are then installed on a regular square grid pattern, with a spacing of 1.0 meter centres in the soft clay layer to ensure an effective distribution of vacuum load and possible discharge of pore water pressures (see Fig 2.6). With the PVDs linked up to corrugated flexible pipes of about 100mm diameter, perforated and wrapped with geotextile filter materials laid horizontally in the sand blanket to connect PVD's to the main vacuum pressure line. Three layers of this PVC membrane are laid to seal each section. A pressure of up to 80kPa is delivered using jet pump and maintained to achieve the desired degree of consolidation. If a vertical pressure greater than 80kPa is required then a combination of surcharge loading and vacuum preloading may be used (Chu et al 2006).

The success of this process is dependent on the effectiveness of field instrumentation; hence, data monitoring using piezometers, settlement gauges and inclinometers are essential to measure the pore water pressure changes, the settlement at ground surfaces and lateral displacements (Chu et al 2006).

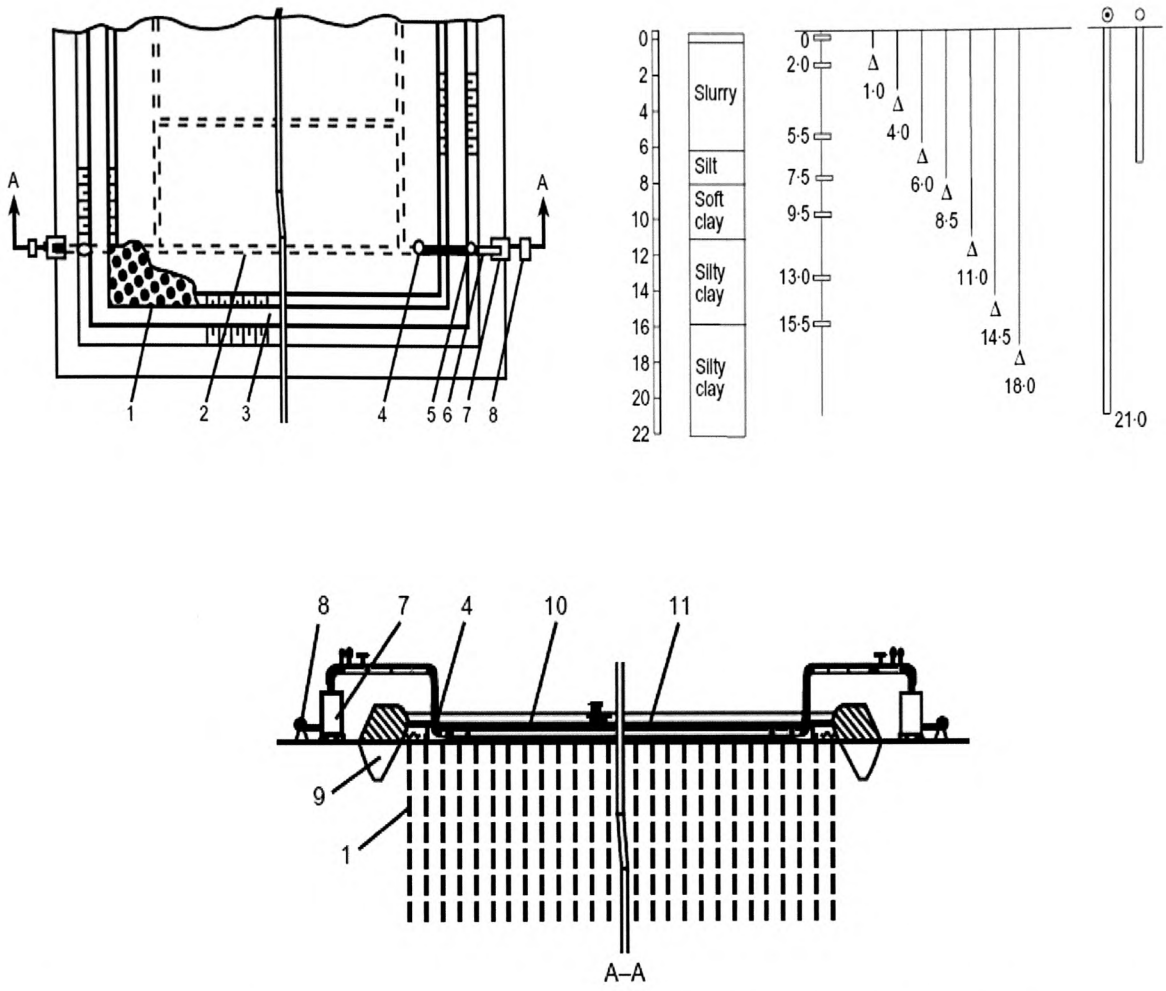


Fig. 2.6 Schematic arrangement of vacuum preloading method: 1, drains; 2, filter piping; 3, revetment; 4, water outlet; 5, valve; 6, vacuum gauge; 7, jet pump; 8, centrifugal pump; 9, trench; 10, horizontal piping; 11, sealing membrane (After Chu et al, 2006)

2.4.2.4.2 Explosive Replacement Technique

Over the years, explosive compaction has been used for the successful compaction of loose granular soils by employing the use of energy from confined detonations of explosive charges placed within the soil mass to densify loose, saturated sand or gravel (Chu et al 2006). However, research has resulted in the development of a method in which soft clay is replaced by crushed stones (known as the explosive replacement technique) used for highways and road embankment constructions (Chu et al 2006).

The basis of this method is to remove soft clay and replace it with crushed stones through controlled blasting. This process is achieved by first introducing charges in the soft clay to remove it. Crushed stones are then heaped on the improved side of the road next to the area

to be improved. By detonating the explosive, the soft clay is blown out resulting in the formation of a cavity. Consequently, the crushed stones collapse into the cavity to form the base of the road, while the soft clay that is blown out forms a liquid and flows away after it falls to the surface. The impact created by the explosive causes an instantaneous reduction in the shear strength of the soil below the level of explosion so that the crushed stones can sink easily into the deeper layer. More of the stones are then placed and compacted to form the final ground profile (Chu et al 2006). There is an overall benefit of cost and time saving, however, from a holistic point of view, there will be agitations especially if project is within a built area. It is advised to carry out campaigns to get people informed before hand as noise will be generated in the processes of blasting.

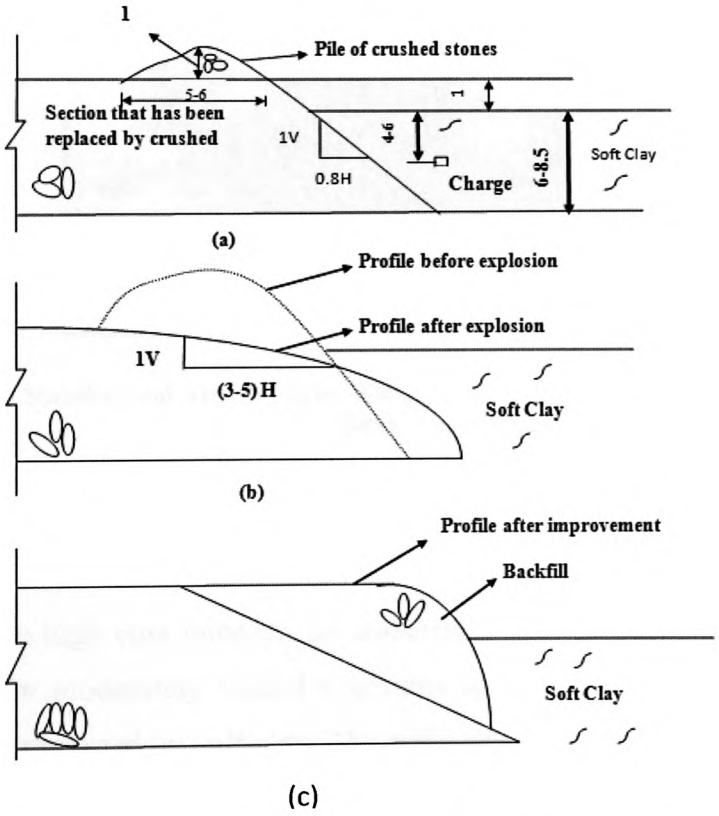


Fig.2.7 Explosive Replacement procedure (a) Before explosion; (b) After explosion (c) After backfill (After Chu et al, 2006)

2.4.2.4.3 Full Depth Reclamation Techniques

This technique which involves deeply mixing of chemical additives into unpaved roadbeds was developed based on the assumption that, unpaved roads which are properly stabilized

through deep mechanical manipulation, can satisfactorily bear traffic for significantly longer periods of time than can un-stabilized roads. The chemical additives used are specifically, calcium chloride, magnesium chloride, etc, which have been investigated to have no undesirable environmental side effects (William et al, 2004). The underlying principle has been the rehabilitation of the full road section and a predetermined portion of the underlying materials by uniformly crushing, pulverizing or blending as a form of in-place recycling of the road pavement. However, chemical additives are mixed into the process, blended as liquids using spraying mechanisms or as powdered stabilizers mixed with soil/aggregate material by help of a reclaiming machine and compacting adequately (William et al, 2004).

Fig. 2.8. Gravel Road Stabilization with Wirtgen WR 2500 Full-Depth Reclaimer (After William et al, 2004).

2.4.2.4.4 Ground improvement by small-diameter timber piles

In other to avert the high cost imposed on construction by problematic soils (soft clayey soils), especially, for moderately loaded structures such as low-to medium-rise buildings often needed to be supported on soft clay. The safe and economic design of such structures often poses problems. However, employing the use of deep piles to avoid excessive settlement may not be of good economic interest, hence, for cost-effectiveness, small-diameter timber piles of 150-200mm diameter and 5-6m long are adopted (Ghosh et al, 2008). This has been an age long practice for supporting low-to medium-rise buildings in and around Kolkata, India (Ghosh et al, 2008). This works on the principle that a relatively hard stratum exists at shallow depth where the pile tips may develop adequate end-bearing resistance (Ghosh et al, 2008).

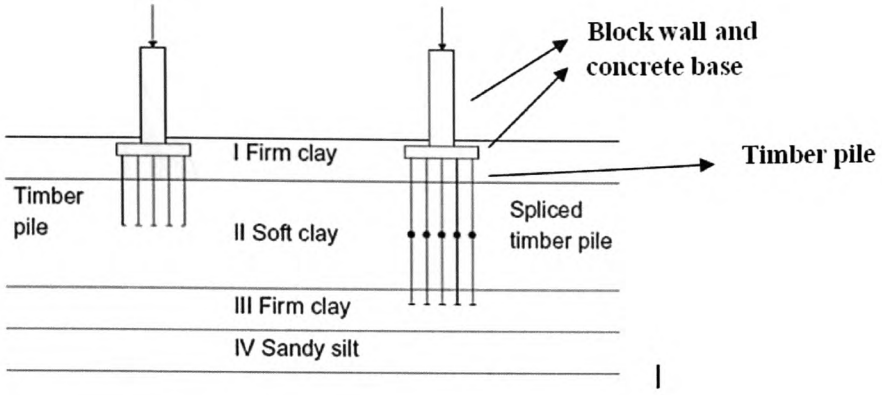


Fig. 2.9. Set-up of small-diameter timber piles and ground profile (After Ghosh et al, 2008).

2.4.2.4.5 Fibredrain and high-energy impact (HEI) techniques.

According to Lee and Karunaratne (2007), the treatment of soft grounds which are major problems to highway construction has been mitigated in the Southeast Asian region by the application of fibredrains and high-energy impact which offers the double benefit of cost-effectiveness and environmental friendliness.

This system is based on checking excessive settlement that might be an aftermath effect of constructing on an expansive soil, by pre-consolidating the clay fraction beneath the soil surface until the anticipated settlement under the envisaged design load is enforced to an acceptable level (Lee and Karunaratne, 2007).

Fibredrains, are vertical drains manufactured with biodegradable jute and coir fibres. They are installed from a sand platform into the compressible clay deposit at a spacing designed to achieve the desired degree of consolidation. The consolidation is targeted to be achieved within the project duration under predetermined surcharge intensity. High-energy impact is then applied by dropping a heavy pounder, ranging normally from 10 to 40 tonnes, from a height usually of about 10-25 meters via a platform of granular fill. By tamping the ground in a regular grid pattern, settlement is enforced on the underlying soft soils, leading to densification in sandy soils and reinforcement in clayey deposits. The only setback with this system is the intensive labour requirement in fabricating the jute fibres and the coconut coir.

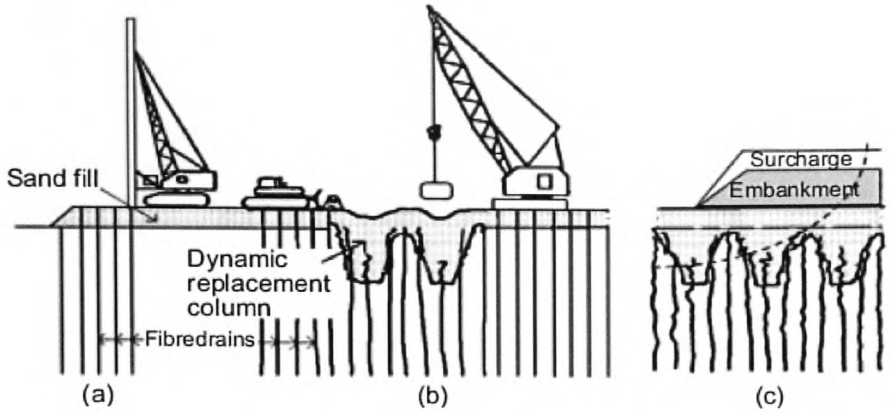


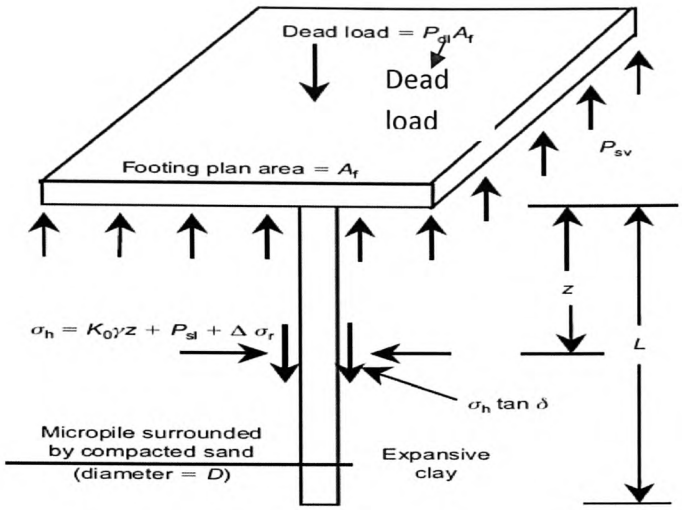
Fig. 2.10. Sequence of soil treatment in embankment construction using fibredrains and high-energy impact (a) pre-fabricated vertical drain installation; (b) heavy tamping; (c) surcharge. (After Lee and Karunaratne, 2007).

2.4.2.4.6 Expansive soil Improvement using micropiles soil reinforcement techniques.

Steel micropiles of 75-250mm in diameter and 3.0m long have been investigate and found to possess the capacity to restrain expansive soils from heaving and causing undue volumetric changes that are capable of inflicting huge maintenance worries to buildings and pavements on the face of seasonal moisture content variations (Nusier et al, 2007).

These micropiles are inserted to predrilled holes of larger diameter than the piles, which are then filled with compacted sand to improve the frictional resistance offered by the micropiles. With the top end of each micropile fastened to the foundation. Upon wetting, a lateral swelling pressure is being generated which adds to the initial horizontal normal stress on the micropile surface. Subsequently, an induced vertical swelling pressure will tend to push the foundation up and in turn pull the micropiles from the surrounding compacted sand thereby mobilising the frictional resistance of the micropiles (Nusier et al, 2007).

According to Sridharam et al (1989) as recorded by Nusier et al (2007), low-frictional-strength soil such as clay, a 15mm thickness of sand around micropile reinforcement is enough to increase the interfacial shearing resistance to that of the sand as the bulk material. Fig. 2.10 shows the mechanism of micropiles for heave control.



K_o = at-rest earth pressure coefficient.

Z = depth below ground surface.

δ = sand-micropile friction angle.

δ_h = lateral earth pressure

Y = unit weight of sand

Fig. 2.11. Mechanism of micropiles for heave control

(After Nusier et al, 2007)

2.4.2.4.7 Soil improvement by internally reinforced stone columns.

Highways and light structures can benefit from the use of stronger gravel-sized stone columns to improve the load-carrying capacity of cohesive soils. This system of improving compressible soils was first used by the French military engineers to support the heavy foundations of the ironworks at artillery Arsenal in Bayonne in the year 1830 (Ayadat et al, 2008).

The process is simple as any method could be used to drill holes and these granular materials poured in, they may sometimes be reinforced by the application of chemical grouting, stabilization using cement plugs or with geofabric. These reinforcements tend to stiffen the column and increase the lateral stresses in the surrounding soil, and accordingly its bearing capacity. This is based on the premise that, the stone column shares the foundation load with the surrounding soil. Hence, under axial loading, the stone column dilates and applies lateral stress to the surrounding soil, creating a high level of passive resistance, which in turn increases the load carrying capacity of the ground (Ayadat et al, 2008). Similarly, sand and gravel columns could be installed in weak soils as well. Andreou et al 2008 proposed the installation of sand and gravel columns in weak soils as a way of improving soil strength. This method is also based on the concept of dilation under

axial loading to increase lateral stress thereby improving bearing capacity and also improving drainage to enhance consolidation.

2.4.2.4.8 Use of Waste Tyre Bales

There are a lot of works and materials relating to soil stabilization, however, very little literature exist in the area of stabilization of soils susceptible to flood inundation. One of those researches dealing with the stabilisation of flooded areas or flood embankments is the work reported by Bo and Yarde (2006) on the use of waste tyre bales to construct a flood embankment (see Figure 2.12). Bo and Yarde (2006) reported the challenges faced in the reconstruction of a flood embankment between River Witham and the Branston Island flood storage area located in Lincolnshire, between Lincoln and Boston near the village of Bardney, where the embankment needed widening and reinforcement to maintain its stability. After dully evaluating all possible stabilisation design options, the team homed in at the choice of using waste tyre bales which possess challenges but proved most viable amongst the different options evaluated. The identified risks in the use of tyre bales in a flood prevention embankment included risk of buoyancy of tyre bales under water, possible piping of underlying soil and overlying capping soils due to their extremely high permeability, possible rebound of the bales upon release or deterioration of metal ties, release of unacceptable leachate to the ground water from old tyres (Bo and Yarde, 2006). The authors concluded that up to date, there has not been any results to prove the adverse effect of the use of tyre bales, although, the programs of monitoring both the chemical and biological implications of this novel project is still ongoing.

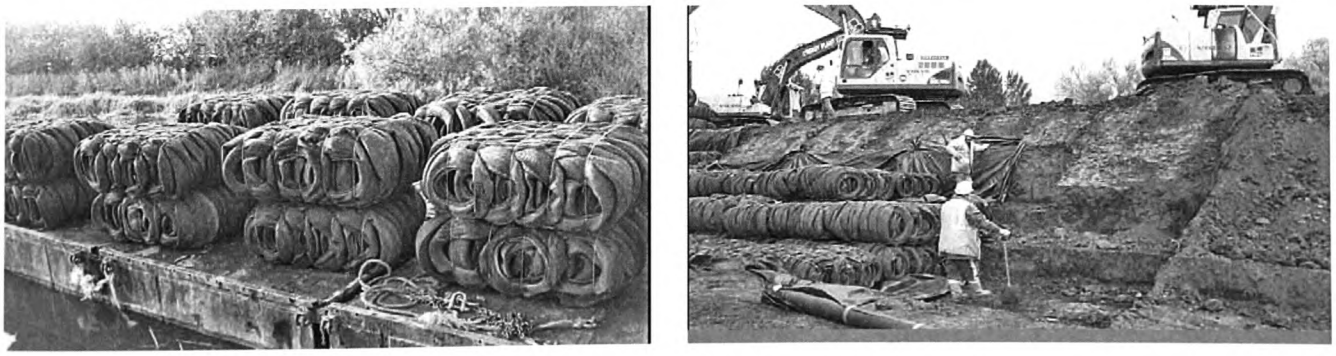


Fig. 2.12 Tyre Bales in flood embankment construction (After Bo and Yarde, 2006)

This soil stabilization option is somewhat limited to highly industrialized areas with high rates of waste tyre production. For other places like some African and Asian countries, the use of tyre bales in soil stabilization could be limited to pilot scale tests otherwise they will represent huge financial cost accruing from import duties and cost of importation.

2.5 Flooding

Flooding according to CIRIA (2004) is not a new problem, as records exist of lives and properties lost in the history of flooding. The flooding of a watercourse's flood plain is a natural process which, in the absence of mitigation measures, may occur periodically, perhaps every year. Similarly, low-lying land along estuaries and coastal areas may naturally flood as a result of heavy storms and high tides and low-lying ground. Flood size is usually described by its statistical return period, e.g. a 50 year flood, which has 2% chance of occurring in any year. This is expressed in terms of flow, stage, height or area (Waltham, 2009). Waltham (2009) identified practices like urbanization, deforestation and levee construction to possess the attribute of raising the height of flood peaks and consequently change floodplain hydrology. The author noted that flood frequency will change with any global warming.

2.5.1 Causes of Flooding

Flooding can result from a wide range of events and processes. The causes of flooding may be as a result of natural or environmental factors, or a combination of both, particularly rainfall, tidal surges and raised groundwater levels (CIRIA, 2004) or from human interference with natural processes, such as changes to river channels and increases in runoff emanating from paved land and blocked drainage systems. According to CIRIA (2004) increased areas of impermeable surfacing and roofing on a new development can increase the volume and rate of surface water runoff during storms, which may exacerbate flooding problems on other sites downstream. Increasingly, development and construction works have almost erased the thin line that existed between flood plains and other none flood susceptible areas, as wide range of locations now stand the risk of flooding as the development of a particular site may increase the flood risk elsewhere (CIRIA, 2004).

2.5.2 Types of Flooding

The level of destruction expected from a flooding event is dependent on the type, source and cause of the flooding. CIRIA (2004) identified different levels of flooding – fluvial, coastal, estuarial flooding and watercourses affected by tide locking, groundwater flooding, overland flow flooding, flooding from artificial drainage systems and flooding from infrastructure failure. Floodplains are at higher risk of these flooding types because of their low-lying grounds, and no flooding event is pleasant as life and properties are lost and enormous economic strain invoked on the economy owing to infrastructural repairs and reconstructions (see Fig. 2.13). In august of 2005, New Orleans was heavily hit by hurricane Katrina, damaging the major roads in and out of New Orleans (Wikipedia) for more related cases see Appendixes E1 and E2 for a register of flooding events in different locations.

Fig. 2.13 Road washed out by a flood in West Virginia.

Source: U.S. Geological Survey Fact Sheet 076-03

2.5.2.1 Fluvial Flooding

This defines the over bank flow from rivers and other watercourses as a result of prolonged or intense rainfall. According to CIRIA (2004) in prolonged high intensity rainfalls, most water holding capacities for most channels are exceeded and this could result in the excess water overflowing the banks to the adjacent floodplains or low land areas and causing widespread flooding (see Fig. 2.14). These flood waters may last for days depending on the prevailing meteorological factors. When the flood waters are short lived, they are then classified as flash flooding. Flash flooding which is so called as a result of the length of

stay of a given flooding event is governed by the type of soil, the level of saturation of the soil, infiltration and drainage characteristics, and the humidity of the surrounding atmosphere. Similarly, situations arise where estuaries and watercourses are flooded by sea water known as Estuarial Flooding and watercourses affected by tidelocking. This is the effect of tidal factors aggravated by fluvial flows. CIRIA (2004) reported that the shape of an estuary channel can cause the high water level to increase as a tide moves up an estuary from the sea.

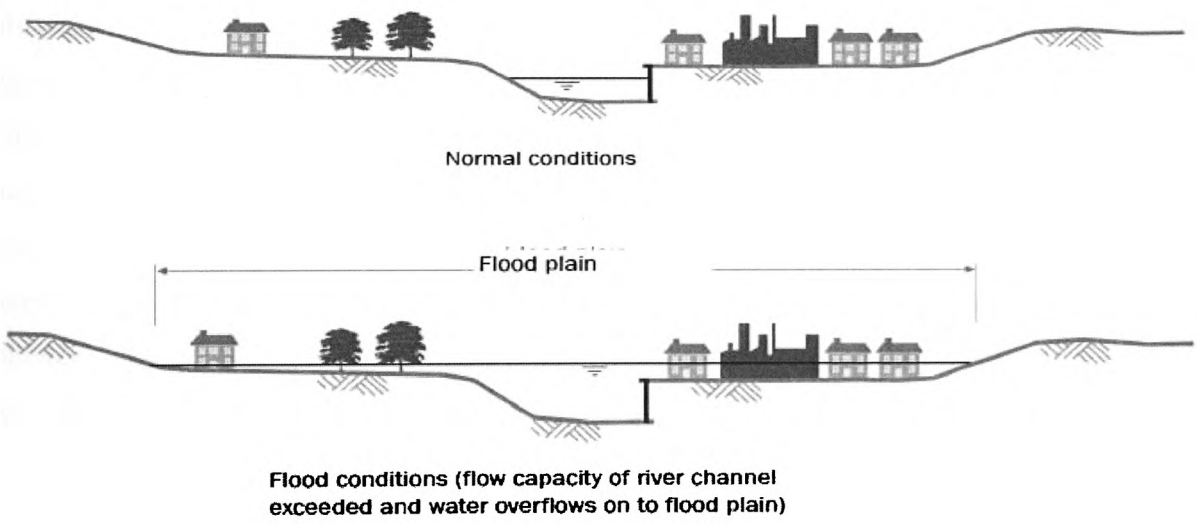


Fig. 2.14 Floodplain and fluvial flooding (After CIRIA, 2004)

2.5.2.2 Coastal Flooding

In extreme weather conditions, the tendency is that high wind results in the occurrence of wave action, surges or high astronomical tide levels on oceans (CIRIA, 2004). When this happens, flood protection works might be surmounted and coastal flooding occurs. This could put millions of people and properties at risk as the adjacent floodplains and hinterlands are heavily hit.

2.5.2.3 Groundwater Flooding

Prolonged rainfall can increase the groundwater level excessively and result in flooding. This form of flooding is characterised by long periods of stay before the floodwaters are drained, and usually occur in places where the underlying materials are impermeable bed rocks (aquifers) (CIRIA, 2004). In the event of heavy rainfall, the aquifers exceed their water holding capacity and the implication is rising levels of underground water since the underlying material is impervious.

2.5.2.4 Overland Flow Flooding

Though low-lying areas are more at risk of this kind of flooding event, overland flows or pluvial flooding can occur anywhere. According to CIRIA (2004) this is a phenomenon where surface runoff for some reasons ponds a given location as a result of blockage of either the natural or artificial water drainage channel. It could also be as a result of heavy down pour in which case the infiltration capacity of the soil is exceeded or soil is saturated such that it cannot take more water in at that given time. This could have localized effects depending on the topography of the location. Coelho et al (2004) reported that this phenomenon could be as a result of water repellency, also referred to as soil hydrophobicity, which could be an effect of reduced soil infiltration capacity and associated increase in surface runoff characteristics.

2.5.2.5 Flooding from artificial drainage systems

To manage runoff from rainstorms artificial drainage systems are constructed – pipes, land drains, sewers and drainage channels (ditches and culverts) etc. During heavy rainfalls the capacity of these drainage systems are sometimes exceeded and this results in flooding (CIRIA, 2004). This type of flooding is usually associated with health risk as they are always polluted with foul water or sewage.

2.5.2.6 Flooding from Infrastructure failure

This type of flooding occurs where infrastructure exists to manage the flow of water – retain, transmits or controls the flow of water. Flooding may be imminent if there is a structural, hydraulic, geotechnical, mechanical or operational failure (CIRIA, 2004). This type of flooding is associated with dam breakages, canal leakage, water mains burst, flap valve failures of flood defence works, pumping station failures and blockages of pipes, bridges or culverts (see Figure 2.15). This tends to have the most severe impact because of the sudden nature and mechanism of flooding.

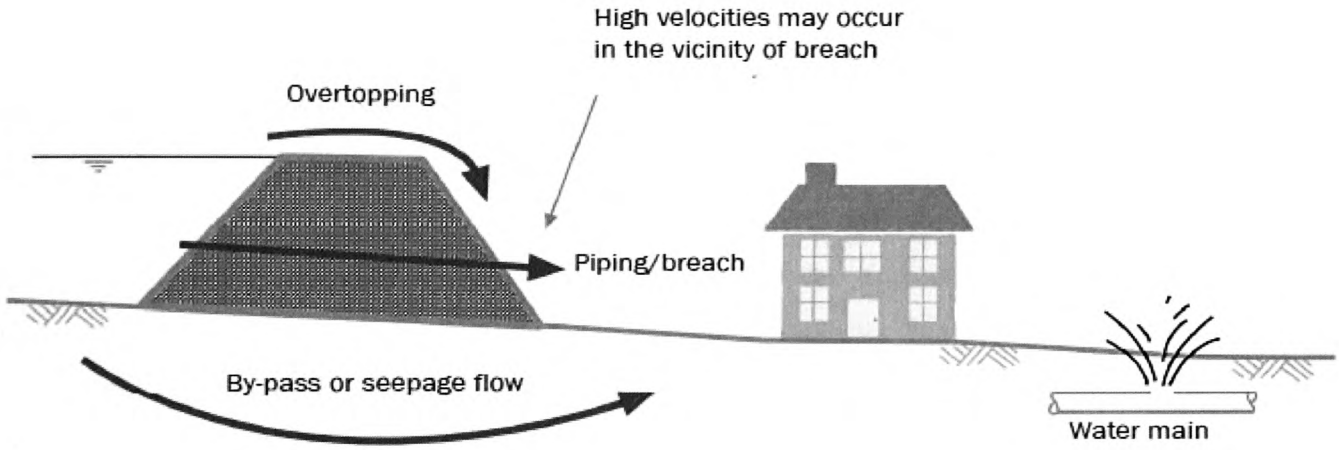


Fig. 2.15 Flooding from infrastructure failure (After CIRIA, 2004)

2.5.3 Impact of floods on roads and Flood Management

There is need for adequate management of flooding and flood waters to avoid undue maintenance cost to infrastructures. Huge maintenance costs are possible following flooding events, as road base and sub-base soil material's carrying capacity can be compromised, leading to premature failure of infrastructures especially roads (Plate, 2002; CIRIA, 2004). The management of flood has become a contemporary issue and flood protection and management can range from local measures, such as temporary solutions – temporarily closing openings with sandbags or brick walls, or just by moving one's belongings to a higher level of the house, to permanent solutions, such as building houses on the high ground, perhaps on artificially generated hills, construction of dykes, dams, levees and other flood defence works (Plate, 2002; CIRIA, 2004; Kelman I., 2007; Akter

and Simonovic, 2005; Simonovic, 1993; Werritty, 2006). According to Plate (2002) there is no technical solution to flooding that is absolutely safe. Even if the system always does what it is supposed to do, it is hardly ever possible to offer protection against any conceivable flood. There is always a residual risk, due to failure of technical systems, or due to the rare flood which exceeds the design flood (Plate, 2002; Akter and Simonovic, 2005). Nevertheless, for effective flood mitigation measures to be designed, there must be an effective forecasting system, which permits the early identification and quantification of an imminent flood to which a population is exposed (Plate, 2002).

Flood risk management in a narrow sense is the process of managing an existing flood risk situation, while in a wider sense it includes the planning of a system which will reduce flood risk (Plate, 2002; Simonovic, 1998). According to Plate (2002) the solution to flood problems is considered a classical engineering task like many others, such as designing a highway or a sewage disposal system. Plate (2002) added that modern options for flood management are not absolute, but depend on three variable factors: the available technology, the availability of financial resources, and the perception of the urgency of the need for protection, which is embedded into the value system of a society.

3.0 Chapter Three: Materials

This research work entails the formulation of a novel material for overcoming the impact suffered by stabilized materials upon flooding. The work was carried out using Lower Oxford clay soil, Lime, Ground Granulated Blastfurnace Slag (GGBS), Portland cement and potable water. Each of these materials will now be discussed in turn.

3.1 Lower Oxford Clay

The Lower Oxford clay used for this research was supplied by Hanson Brick Company Ltd., from their Stewartby brick plant in Bedfordshire UK. The mineralogical composition is shown in Table 3.1 while the physical and chemical properties are shown in Table 3.2. On receipt of the material, it was air dried to reduce excess moisture content and stored in the laboratory in polythene bags as a measure of moisture content control. The laboratory environment provided a medium for climate control; the drying became imperative to ensure that subsequent moisture additions were under regulation to achieve the predetermined working moisture content level. The choice of Lower Oxford Clay (LOC) was driven by the fact that when the clay is stabilized with lime, it exhibits some swelling tendencies in the presence of excess moisture, coupled with the sulphate bearing nature of the soil. Secondly, if the unpredictable behaviour of this material could be contained, then, any other scenario presented by any other soil could be easily addressed.

A good number of investigations have been carried out on Lower Oxford/Kimmeridge clay for different reasons and different results have been achieved likewise. Wild et al (1998) reported the investigation carried out on LOC by stabilizing it with varying percentages of Lime replaced with GGBS. This material upgrading was supposedly mitigating the global environmental problems of pollution and natural resource depletion. It was concluded that the replacement of Lime with GGBS resulted in strength improvement when stabilized with LOC. This has the implication of cost saving and environmental conservation. Similarly, Wild et al (1999) investigated the swelling potentials of sulphate bearing LOC resulting to the formation of ettringite upon stabilization with lime. It was uncovered that the partial substitution of lime with GGBS between 60 – 80% resulted in improved durability and reduced swell potentials. In an attempt targeted at reducing the cost and environmental effects of firing clay to produce bricks, Oti et al (2008) reports on the

stabilization of LOC with Lime blended GGBS at a high stabiliser level of 20% to avert firing. The authors concluded that there is potential for using blended stabilizer for the manufacture of unfired clay materials within the building construction industry. Also, Kinuthia et al (2004) reported the stabilization of LOC with a blend of traditional stabilizer (Lime) with Wastepaper Sludge Ash (WSA). The authors noted that higher strength values and volume stability were achievable with a blend of this novel material with small quantities of CaO. In all, the expansive tendencies possessed by this soil have always offered reasonable challenge to many researchers.

Table 3.1. The mineralogical composition of Lower Oxford Clay used in this research (Hanson Brick Company Ltd.)

Compound	Chemical formula	Composition (%)
Illite	$(K,H_3O)Al_2Si_3AlO_{10}(OH)_2$	23
Kaolinite	$Al_2Si_2O_5(OH)_4$	10
Chlorite	$(OH)_4(SiAl)_8(Mg.Fe)_6O_{20}$	7
Calcite	$CaCO_3$	10
Quartz	SiO_2	29
Gypsum	$CaSO_4.{}_2H_2O$	2
Pyrite	FeS_2	4
Feldspar	$CaAlSi_3O_8$	8
Organic materials	-	7

Table 3.2. Chemical and Physical properties of Lower Oxford Clay used for this investigation (Bari Thomas, 2001)

Oxide	Composition (%)
CaO	6.15
SiO_2	46.73
Al_2O_3	18.51
MgO	1.13
Fe_2O_3	6.21
MnO	0.07
TiO_2	1.13
K2O	4.06
FeO	0.8
P_2O_5	0.17
Na_2O	0.52
Others	14.52
Consistency Limits	
Liquid Limit (LL)	65
Plastic Limit (PL)	36
Plasticity Index (PI)	29
Colour	Grey

3.2 Lime

The lime used for this research is quicklime (Calcium Oxide (CaO)), this was supplied by Tŷ – Mawr Lime Ltd., Llangasty Brecon, UK. The chemical and physical properties are shown in Table 3.3. The use of lime as a stabilization material has been well established (BRE, 1998; Howe, 1974; Puppala et al, 2003). Having been exhaustively investigated by a pool of researchers to validate varying research interests, lime is found to give its utmost performance with high plasticity soil types (Ellis, 1974). In all, its self-healing property has always favoured most stabilization applications (Higgins D. D., 2005; Little et al, 2000). However, for the sake of this study, its readiness to engage in an exothermic reaction with soil in the presence of moisture to liberate water in the form of steam cannot be over emphasized, as this process leaves the material drier with improved strength and enhances usage for engineering applications (Bell, 1996). Lime is also known to posses the quality of involving the pozzolans in a long term reaction, with the possible benefit of strength improvement and reduction in plasticity (Bell, 1996).

The manufacture of lime involves the heating of the raw material (limestone, chalk, marble, coral etc) with $CaCO_3$ as an essential component, to calcination temperatures of between 800°C to 1200°C in a kiln thereby disintegrating the calcium carbonate compound to liberate CO_2 (BRE, 1998; Ellis, 1974) as shown below. However, precautionary measures are essential to avoid the recombination of the resultant oxide with carbon dioxide upon cooling (BRE, 1998). In the interest of soil stabilization, calcium oxide (quicklime, CaO) and Calcium hydroxide (slaked lime, $Ca(OH)_2$ are the two forms in which lime could be applied to the soil.

$$\mathbf{CaCO_3 + Heat \rightarrow CaO + CO_2} \quad \text{.....Eq 3.1}$$

Table 3.3. The Chemical and Physical properties of Lime (Calcium Oxide) used for this investigation

Oxide	% Composition
CaO	89.2
SiO_2	3.25
Al_2O_3	0.19
MgO	0.45
Fe_2O_3	0.16
MnO	0.05
S^{2-}	<0.01
SO_3	2.05
SO_4	2.46
K_2O	0.01
N_2O	0.02
Soluble Silica	1.1
Others	
CO3	4
Free Lime	51.1
Insoluble Residue	4.1

3.3 Ground Granulated Blastfurnace Slag (GGBS)

The Ground Granulated Blastfurnace Slag (GGBS) used for this study was supplied by Civil and Marine Slag Cement Ltd., Llanwern, Newport, UK. Its physical and chemical properties are shown in Table 3.4. GGBS is an industrial by-product from the manufacture of pig iron in the blast furnace. The production is a combination of iron ore with limestone flux, the slag in its molten form is tapped from the kiln and cooled by the process of rapid quenching with water. This process is commonly known as granulation (Higgins, 2007); the GGBS solidifies to small granules of glassy appearance. This process is known to influence the physical characteristics of the emergent material while the chemical composition and consistency is a function of the ore and the heating process (temperature). After the quenching, the granules are then ground to form the GGBS used for construction. The use of GGBS is still at its infancy stage especially for soil stabilization applications. However, in the arena of cement/concreting, its enhanced durability and resistance to high sulphate attacks properties has been reported by Wild et al (1999) and Higgins (2005) among other researchers. Nevertheless, available literature proves also that GGBS activated with lime offers revolutionary stabilization technology in the stabilization of

sulphide and sulfate bearing clay soils (Higgins, 2005; Wild et al, 1999; Puppala et al, 2003). Similarly, extensive research work carried out by Wild et al (1998) demonstrated the viability of GGBS in combating expansion of sulphate-bearing clay soils by arresting the formation of ettringite. This characteristic makes up for the shortcoming of most traditional stabilizers.

Table 3.4. Chemical and Physical properties of GGBS used for this investigation (Civil and Marine Slag Cement Ltd. UK)

Oxide	Composition (%)
CaO	41.99
SiO_2	35.35
Al_2O_3	11.59
MgO	8.04
Fe_2O_3	0.35
MnO	0.45
S^{2-}	1.18
SO_3	0.23
Insoluble Residue	0.3
Bulk Density (kg/m3)	1200
Colour	Off-white
Glass Content	90

3.4 Portland cement (PC)

The Portland cement used for this investigation was supplied by Lafarge cement UK. The mineralogical composition is presented in Table 3.5 while its physical and oxide compositions are shown in Table 3.6. The use of cement as a binder is well established (Neville, 1995) and this study employed its use as a control to measure against the performances of other not well known binders. However, not much work was done with the Portland cement as this research aims to reduce the over reliance on traditional stabilizers – the use of Portland cement and lime as a way of promoting the sustainability agenda and subsequently cut down on energy usage, carbon dioxide emission and resource depletion. As noted by George, (1974), "now that the energy crisis is upon us, the need to develop ways of doing things that minimise the use of non-renewable sources of energy is becoming increasingly urgent in the industrialised countries, as well as in the developing world. Work on capital-saving technology is becoming a necessity for survival all over the world".

According to Bye (1999) and Little et al (2000) cement is a material which binds together solid bodies by hardening from a plastic state, this develops rigidity (sets) and then steadily increases in compressive strength by a chemical reaction with water otherwise known as hydration – this is typical of hydraulic cement which develops strength even under water (Eglinton, 1987). There are two established processes in the manufacture of Portland cement, the wet and the dry processes (Bye, 1999; Neville, 1995). Whichever process is applied in cement production, the overriding issue is that being subjected to a temperature of the range of 1350°C to 1500°C, enormous energy is expended (Robertson, 1974; Higgins, 2007), and large volumes of green house gases are produced amidst the large amounts of raw materials utilized (Bye, 1999; Neville, 1995; Higgins, 2007). Higgins, (2007) has reported that 1.5t of limestone and clay are quarried to make 1t of cement.

Table 3.5. The mineralogical composition of Ordinary Portland cement used in this research

Compound	Composition (%)	Chemical Formula
Tricalcium Silicate, C_3S	50	Ca_3SiO_5 or $3CaO.SiO_2$
Dicalcium Silicate, C_2S	25	Ca_2SiO_4 or $2CaO.SiO_2$
Tricalcium Aluminate, C_3A	10	$Ca_3Al_2O_6$ or $3CaO.Al_2O_3$
Tetracalcium Aluminoferrite, C_4AF	10	$Ca_4Al_2Fe_{10}$ or $4CaO.Al_2O_3.Fe_2O_3$
Gypsum	5	$CaSO_4.2H_2O$

Table 3.6 The chemical and Physical properties of Ordinary Portland cement used in the research

Oxide	Composition (%)
CaO	63
SiO_2	20
Al_2O_3	6
MgO	4.21
Fe_2O_3	3
MnO	0.03-1.11
SO_3	2.3
Insoluble Residue	0.5
Bulk Density (kg/m^3)	1400
Colour	Grey

3.5 Water

Water is a universal solvent and the most abundant molecule on the Earth's surface, constituting about 70% of the planet's surface. In nature its existence is in liquid, solid and

gaseous states. It is a nearly colourless, tasteless and odourless substance at standard temperature and pressure (Wikipedia). Potable water devoid of any contamination and complying with the standard for drinking water as recommended by the World Health Organisation (WHO, 2008) was used for the whole of this investigation. The chemical and physical analyses as stipulated by WHO, are presented in Table 3.7. The use of water fit for drinking became paramount to avoid any possible contamination emanating from water sources. Although, in nature this could not be guaranteed always, as subsurface water movement cannot be completely eliminated. This has the tendency of introducing sulphates to most foundation applications. The introduction of sulphates to different sub surface applications by underground water has been the subject of different investigations.

Water is formed by the reaction between hydrogen and oxygen as follows:

$$2H_2 + O_2 \rightarrow 2H_2O$$

Water being a polar substance dissociates disproportionately into the hydonium ion (H_3O^+ (aq)) and an associated hydroxide ion (OH^- (aq)) thus:

$$2H_2O(l) \leftrightarrows H_3O^+(aq) + OH^-(aq)$$

The aqueous OH^- ion produced when water dissociates is the bed rock for most reactions involving water, making it a universal solvent.

Table 3.7 The Chemical and Physical analyses of the water used for this research (WHO, 2008)

Property	Composition
Molecular Formula	H_2O
Molar Mass	18.01528(33) g/mol
Appearance	White solid or almost colourless
Density	1000kg/m^3, liquid (4°C)
Melting Point	0°C, 32°F (273.15K)
Boiling Point	100°C, 212°F (373.15K)
Acidity (pka)	15.74
Basicity (pkb)	15.74
Refractive Index (n_D)	1.333
Viscosity	0.001 Pas at 20°C
Crystal Structure	hexagonal
Dipole Moment	1.85D
Main Hazard	Drowning

Note: All data are given for materials in their standard state (at 25°C, 100kpa)

Chapter Four: Research Methodology

In this chapter, all the experimental procedures used in the different investigations are elaborately explained. The experimental procedures of interest are:

1. The consistency (Atterberg) limits –Liquid and plastic limits; Plasticity index.
2. Sample preparation
3. Curing procedures
4. Proctor compaction test – Optimum Moisture Content (OMC); Maximum Dry Density (MDD),
5. Strength
6. Durability Tests
7. Analytical Tests.

4.1 Consistency (Atterberg) Limits

The consistency or Atterberg limit tests are named after Albert Atterberg a Swedish chemist (March 19, 1846 – April 4, 1916) who founded them, though the tests as they are currently carried out are as refined by Arthur Casagrande and published in 1932. The tests in the current work were carried out in compliance with the provisions of BS 1377 part 2 of 1990 (EN 1997-2:2007) which is the classification tests part 2 of the British Standard Methods of Test for Soils for Civil Engineering purposes. The target material was Lower Oxford Clay (LOC) passing sieve 425μm. This was oven dried for up to 24 hours at a temperature of between 105°C to 110°C as prescribed by the standard. This ensured that the excess moisture contained in the material was driven off.

4.1.1 Liquid Limit

BS 1377 part 2 of 1990 defines liquid limit as the empirically established moisture content at which a soil passes from the liquid state to the plastic state. The liquid limit of the Lower Oxford Clay soil was carried out on dry material passing sieve 425μm and weighing about 300g. The weighed material was placed on a flat glass plate of 10mm thickness and about 500mm square. Distilled water was then added to the material and with the aid of two palette knives, the material was mixed thoroughly with the water for a period upwards of

10 minutes so as to achieve a consistent appearance. Using the palette knifes, a brass cup of about 56mm in diameter and 41mm deep was carefully filled with the material while ensuring that there was no air entrapment, in compliance with BS 1377 (1990). The surface was flattened and excess soil removed with the aid of the straight edges of the palette knife, giving a smooth level surface. The full cup was then placed under the penetration metal cone (angle 30±1°C and 35mm in height) locked in a raised position and the supporting assembly lowered so that the tip of the cone just touched the surface of the soil. The stem of the dial gauge was lowered to make contact with the cone shaft and the reading of the dial gauge and scale was read off to the nearest 0.1mm. At the lapse of five seconds the cone was locked automatically and the gauge read off. It was then wiped clean of soil stains. A soil sample of about 10g was collected from the zone of penetration for the determination of moisture content in accordance with BS 1377 (1990). The remaining soil in the cup was then removed and remixed with that on the glass plate at higher moisture content. This process was repeated, recording the different penetration measurements against their corresponding moisture content values, ensuring that a difference of not more than 0.5mm was established between two subsequent values. A penetration range of 5 – 30mm was established and plotted on a graph against the corresponding moisture contents. The liquid limit was determined from the graph as the moisture content at the point that corresponded with a penetration value of 20mm. Plate 4.1 shows the apparatus and assembly for the determination of liquid limit.

Plate 4.1 apparatus for the determination of Liquid Limit (Consistency Test)

4.1.2 Plastic Limit

The Plastic Limit of a soil as defined by the British Standard, BS 1377 part 2 of (1990), is the empirically established moisture content at which a soil becomes too dry to be plastic. During the determination of the liquid limit of a sample, some material was set aside for the determination of the plastic limit. About 20g of soil was placed on the glass plate and rolled to the shape of a ball. The ball of soil was then placed between the two palms of the hand and rolled until 6mm diameter rods were achieved. The heat of the hands dried the soil sufficiently for slight cracks to appear on its surface as stipulated in the standard. This was then cut into shorter lengths and rolled in turns between the fingers and the glass plate to rods of about 3mm in diameter, while carefully observing the point at which the soil rods sheared longitudinally or transversely. The crumbled pieces were used for Plastic Limit moisture content determination according to BS 1377 part 2 (1990).

4.1.3 Plasticity Index

After determining the liquid (ω_L) and plastic (ω_P) limits, the plasticity index according to BS 1377 part 2 of (1990) is determined as the difference between the two empirical parameters as follows:

$$I_p = \omega_L - \omega_P \quad \text{.....(4.1)}$$

Where Ip is the plasticity index, ω_L is the liquid limit, and ω_P is the plastic limit respectively.

4.2 Proctor Compaction Test

The relationship between moisture content and dry density of soils is significant as the bearing capacity of any soil is controlled by the ratio of the three components of soil – solid matter, water and air void, and these determine the engineering properties of any soil. To achieve optimal performance from soil, it is the aim to compact the soil at its optimum moisture content (OMC), as at this moisture content value the soil yields the maximum dry density possible. BS 1377 part 4 of (1990) prescribes the method for carrying out this compaction test using 2.5kg and 4.5kg rammers depending on the density to be achieved. This results in to two degrees of compaction. A third method relates to vibration test, especially for cohesionless or granular soil types.

In order to determine the OMC and MDD of the Lower Oxford Clay, the 2.5kg rammer option was applied to achieve the required compaction energy, in compliance to the provisions of BS 1377 part 4 of (1990). Dry soil weighing about 300g and passing 20mm sieve was mixed with a given amount of water and compacted in three equal layers in a steel cylindrical mould. To each layer 27 blows of 2.5kg each were administered using a motorized compactor (see Plate 4.2.) The cylinder was fitted with a collar which was removed after compaction and the excess soil above the cylinder which was ensured not to be more than 6mm high (as per BS 1377) scrapped off and the whole assembly (mould + compacted soil) weighed and recorded as (w_2). The empty cylinder and the base were weighed together and recorded as (w_1). With a known volume of cylinder (V), the dry density of the soil (ρ_d) was worked out using the following formulae:

$$\rho_d = (w_2 - w_1)/V \quad \text{.....(4.2)}$$

The soil was then extruded from the cylinder and broken up. A little amount of soil was taken out from the inner parts of the soil core for moisture content determination in compliance with BS 1377 part 2 of (1990). The residual soil was then remixed with additional material after increasing the moisture content. This was carried on for up to five consecutive times and the different dry densities recorded against their corresponding moisture contents. A graphical plot of the dry densities against their corresponding moisture contents gave a parabolic curve from which the summit of the line of best fit plotted through all the points formed the maximum dry density, MDD and the moisture content corresponding to this point was read off as the optimum moisture content. These

parameters formed the baseline values on which future assessments were based, so as to achieve the best compaction efforts and strength development for the different soil-stabilizer blends investigated.

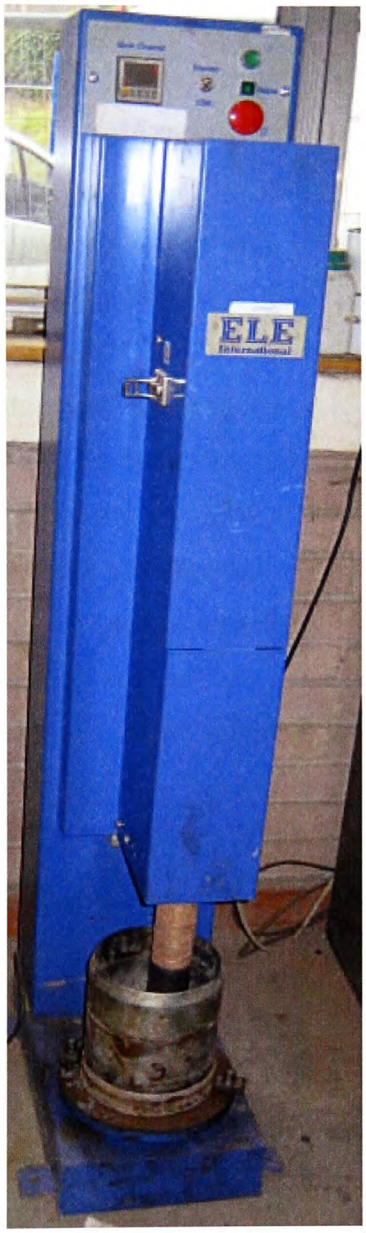


Plate 4.2 - Proctor compaction test machine – for MDD and OMC determination

4.3 Sampling/Sample Preparation

Lower Oxford clay soil was the target material for this study. The entire experimental work was carried out on one supply or batch of the material, so as to enhance consistency of results. The cylindrical samples measuring 50mm in diameter and 100mm high (see Plate 4.3) which were used for all the different experiments set up as described in this chapter were produced from the dried soil stabilized with varying dosages of lime, PC, and GGBS as control stabilisers, together with lime – GGBS and PC – GGBS stabilizer blends. This was carried out at different moisture content levels, starting with a basic moisture content determined using the proctor compaction test described earlier. Having known the Optimum Moisture Content (OMC) and the Maximum Dry Density (MDD) of the clay material, it was possible to manipulate the material mixture within a range of moisture content values above the OMC of the clay material, with a view to achieving optimum performance. Hence, moisture contents of 23%, 28%, 33% and 38% were adopted. Compaction was achieved by applying a static compression pressure using a hydraulic jack assembly as show in Plate 4.4. Before compaction, enough material to produce one cylindrical sample was weighed, mixed with a predetermined amount of water and stabilizer material, and poured into a cylindrical steel mould fitted with steel collars (see Plate 4.5). For each experimental regime, three specimens using each of the different stabilisers blends, were produced and average parameters calculated from the three test specimens. This helped in reducing the risk of experimental errors, while values with difference more than (20%) variation from the mean of the closest two tests were discarded.

Plate 4.3 - Typical cylindrical test sample

Plate 4.4 - Set up for Static compression using a hydraulic jack assembly

Plate 4.5 - Sets of Cylindrical Moulds, Steel Plungers and Steel Collar Assembly

4.4 Curing Conditions

On production of the specimens, they were wrapped with cling films to regulate changes in moisture content between the specimens and their environment, especially, within the early stages of strength development. The development of strength is usually through the hydration process and moisture is critical to this process. Except for the samples meant for linear expansion that were stored in the linear expansion perplex chambers under moist curing conditions, other samples were stored in plastic containers (see Plate 4.6) and kept in a climate regulated environment. A temperature range of 20°C ±2°C was considered appropriate as this was near room temperature and required no heating or cooling. It could also facilitate hydration reactions, and being sealed with cling film promoted high humidity within the curing environment as well as preventing excessive temperature losses. According to Little et al (2000) curing is important for chemically stabilized soils and aggregates – particularly lime-stabilized soils – because lime-soil reactions are time and temperature dependent and continue for long periods of time (even years).

Plate 4.6 - Samples under curing conditions

4.5 Unconfined Compressive Strength

The use of unconfined compressive strength (UCS) test in assessing the strength viability of stabilized clay materials for use in road sub-grades and sub-bases is of contemporary origin (Bari Thomas, 2001; Little et al, 2000). This is aimed at refining the traditional process of employing the empirical practice of California Bearing Ratio (CBR) test that is more widely established. For a laboratory based work of this kind, the use of unconfined compressive strength test is justified as material usage is drastically reduced by over 80% relative to the CBR process. Although there have not been any direct correlation between the two parameters, UCS tests have proven to be a good assessment criteria for validating the carrying capacity of stabilized clay materials.

The samples for unconfined compressive strength (UCS) test were tested at the lapse of different curing ages which for the current research work were 7, 14, 28, 56 and 90 days. These were carried out at a strain rate of 2mm per minute, using a Hounsfield Compression Testing machine (see Plate 4.7), capable of loading up to 10kN. As well as monitoring compressive strength, the machine was also used to record the compressibility of the samples at given load applications through an extension monitoring device that is electrically operated and attached to the compression testing machine.

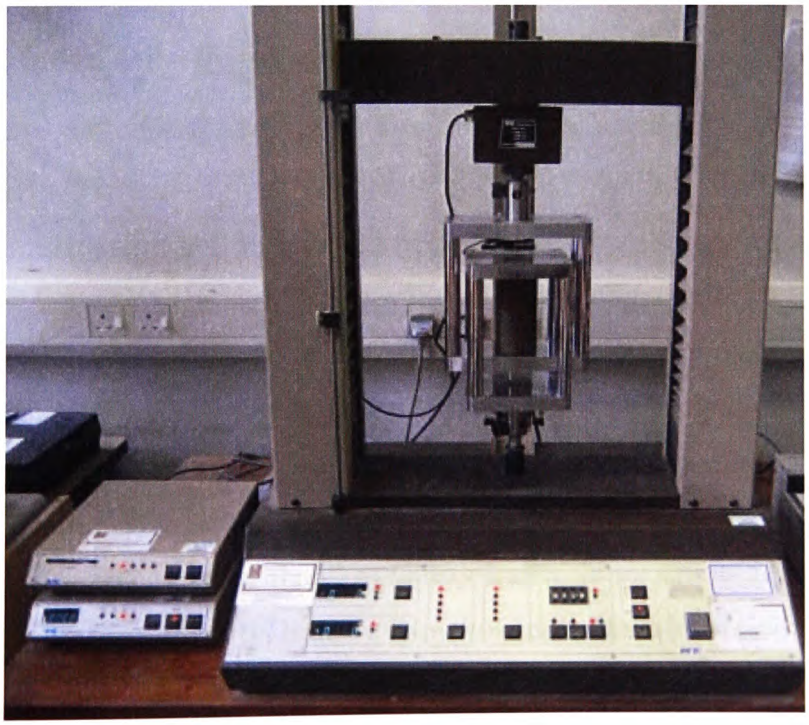


Plate 4.7 - A 10,000N capacity Hounsfield Compression Testing machine and strain monitor

4.6 Durability Test

Owing to the growing concerns of natural resources depletion, more attention is being channelled to ensuring an extended robustness and improved durability for construction material systems, as a way of carrying forward the durability agenda. Stemming from this, the Transport Research Laboratory (TRL) developed an empirical method of assessing durability known as the Durability Index (DI) whereby, samples were soaked in water and assessed against the strength of normal samples that were not soaked in water. The loss in strength resulting from the soaking process was expressed as a percentage of the soaked sample strength against unsoaked sample strength. The TRL conducted this test on samples for 28 days strength, whereby the soaking regime started on the twenty-first day of curing to achieve a seven days soaking at the lapse of a 28 days curing period. In the current work, two regimes of soaking were established, a four days and ten days soaking regimes. On production of the samples, they were wrapped in cling films and cured for different periods of 7, 14, 28, 56 and 90 days before subsequent soaking in water. At the lapse of the different soaking periods, the samples were tested using the unconfined compressive strength method, meanwhile, another set of samples were allowed to complete the same cycle of curing period, however, these were unsoaked. The loss in strength of the soaked samples were calculated thus:

$$\textbf{Durability Index, DI} = \frac{\textbf{UCS Value after Soaking}}{\textbf{Original Strength Value (unsoaked strength)}} \times 100 \quad \text{.....(4.3)}$$

Having identified different degrees of flooding during the review of literature, it was necessary to formulate a system that would be durable enough to withstand flash flooding which could last between one and four days as well as other kinds of flooding such as fluvial flooding which last well above seven days. The research considered it worthwhile to carry out the soaking process for about ten days while monitoring other parameters other than strength. However, this research defined different soaking regimes – four days complete soaking, four days partial soaking and ten days complete soaking (see Plate 4.8). In the durability assessment, other than Durability Index, other durability assessments that were considered were permeability, linear expansion and water absorption tests.

Plate 4.8 - Four days partially and completely soaked samples

4.6.1 permeability Test

According to Basheer et al, 2001, permeability is that property of a fluid medium which characterises the ease with which a fluid will pass through it under the action of a pressure differential. Similarly, Darcy's law states that the steady-state rate of flow is directly proportional to the hydraulic gradient, expressed mathematically thus:

$$v = Q/A = -K(dh/dL) \quad (4.4)$$

$$I = (dh/dL) \quad (4.5)$$

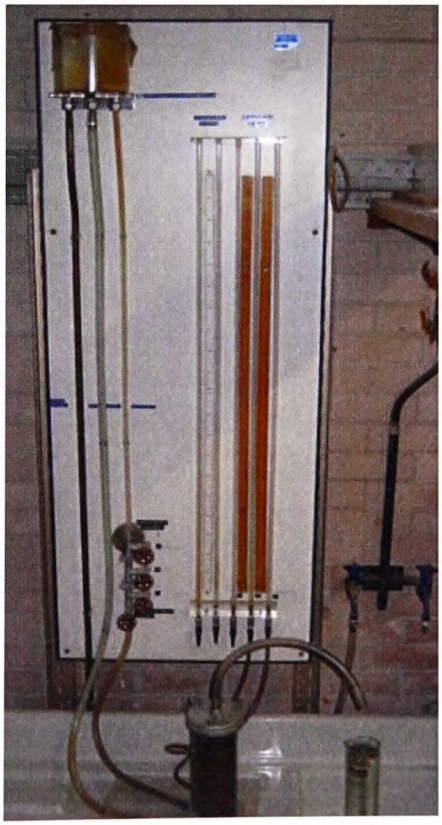
Where v is the apparent velocity of flow, Q is the flow rate, A is the cross-sectional area of flow, I is the hydraulic gradient and dh is the head loss over a flow path of length dL. K is the coefficient of permeability. In practice, permeability measurements are disaggregated into two major divisions. Those dealing with the testing of materials in-situ and the laboratory based measurements. For the sake of the current research we shall explore the later group in more details.

The economic impact of the durability problems has an implication of huge economic costs owing to extensive repairs (Basheer et al, 2001; Puppala et al, 2003), hence, the usefulness of permeation tests to assess the durability of cementitious materials is important, as it is going to reveal the extent of hydration, the rigidity or compactness of the samples and the structural robustness of the formulation in adverse weather conditions of flooding. According to Basheer (2001) any long-term performance depends on the interactions with the service environment, in which the penetration of deleterious substances is highly significant. The author further asserted that, permeability is influenced by the pore size

distribution and continuity of the pores within a given system. However, Hall et al (1997) noted that, cement-based materials are complicated, mineralogically, chemically and micro-structurally; above all they are chemically active. Because of their chemical activity, liquid flow through cementitious material is different from flow through, say, the sand-column of Henry Darcy.

In conventional practices the falling head and the constant head permeability tests are used for the assessment of soil permeability in the laboratory (see Plates 4.9a and 4.9b), for both cohesionless and cohesive soils. Also in concrete, the laboratory assessment of permeability could range from empirical assessments like Initial Surface Absorption Tests (ISAT) (BS 1881 part 5 of 1970 methods of testing concrete for other than strength, to more rigorous practices like the Autoclam air permeability test (Basheer et al, 2001). However, for a cementitious stabilized soil material where strength is of concern relative to concrete structures and with sample structure not conforming to the established testing standards, a compromise should be sought in both the experimental set up and the available testing method. Hence, the decision to tailor the current research endeavour to accommodate the short falls in our specimen meeting the requirements of traditional testing apparatus or developing new testing schemes. In the current work, graduated tube of about 50cm^3 opened at both ends with a locking device fitted to one end and hung up high at the other end was used. This was connected to a cap of about 50mm in diameter made from Perspex material and its inner surface sealed with a gasket to prevent water leakages. This was mounted in a Perspex confinement with holding down bolts. The cylindrical specimens were then introduced and the tube subsequently filled with water (see set up in Plate 4.10). Having surmounted any possible occurrences of water leakages, the samples were monitored for 7, 14, 28, 56 and 90 days to ascertain the rate at which water drains through the compact stabilized materials with time. These were monitored as a draw down on the graduated tube and carried on till the flow of water was seen to be close to constant. Then data were collected, plotted and analyzed.

(a) - Falling Head Permeability Test set up



(b) - Constant head permeability test set up

Plate 4.9 Traditional Laboratory-based Permeability Testing Apparatus.

Plate 4.10 - Newly developed Perspex Confined Permeability testing Apparatus

4.6.2 Linear Expansion Test

Cylindrical test specimens 50mm diameter by 100mm long were produced and immidiately wrapped with a few runs of cling film to regulate moisture movement. A few minutes were left to elapse to enable the samples to relax and avoid any rebounding effect which would have occurred from the release of the compactive pressure as this would have induced a false expansion. With both ends of the cylindrical specimens exposed, the specimens were carefully installed in the automatic linear expansion Perspex cells where axial movements of the specimens were monitored automatically during the hydration period with the aid of transducers. This expansion monitoring process started with seven days period of moist curing and extended to a soaking period whereby the specimens were soaked in water up to 10mm above their bases. This lasted for 43 days bringing the total curing period to 50 days during which linear expansion was monitored. Within this period, the transducers were able to detect any axial displacements, which were set to pick up expansion signals twice in twenty-four hours and transmitted via the different channels of the linear expansion apparatus to an MPX 3000 data logger. The data could be accessed for plotting and analysis and reporting, by means of WINHOST software. The experimental set up is shown in Plate 4.11. This process was also repeated for the mellowed samples, the only variation being that the specimens were passed through a mellowing regime. Mellowing involves the addition of predetermined quantities of lime and water mixed with the clay material and left for 24 hours before the final mixing and compaction. This prepares the clay material for the introduction of cementitoius stabilizers to initiate hydration reactions.

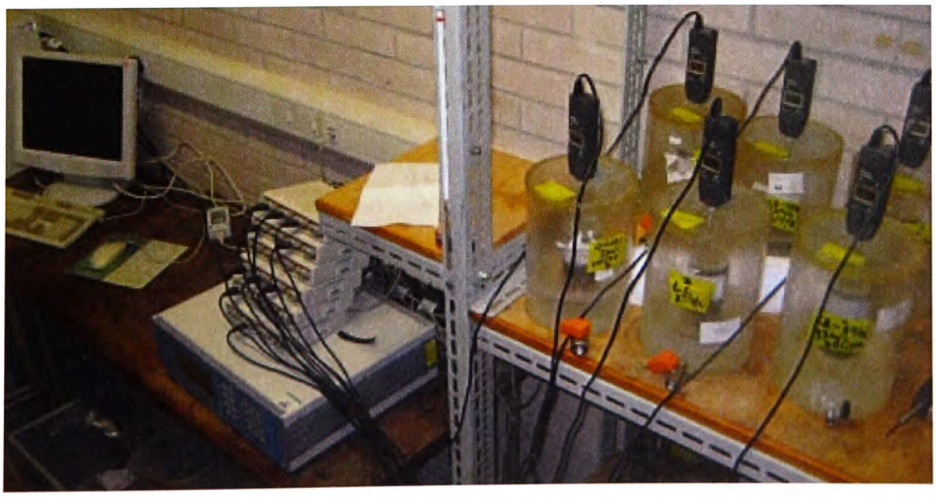


Plate 4.11 Experimental set-up for Linear Expansion measurements

4.6.3 Water Absorption Test

Basheer et al, 2001, defined water absorption as the mechanism by which liquids are transported in porous solids due to surface tension acting in capillaries. They further identified two basic parameters for consideration in the water absorption of cementitious mediums - the mass of water which is required to saturate the cementitious material (the effective porosity) and the rate of penetration of the capillary rise (the sorptivity). In the current research, 50mm diameter and 100mm long cylindrical specimens were produced and cured for different curing periods. At the end of the curing period of 7, 14, 28, 56, 90 and 120 days samples were unwrapped of cling film and completely immersed in water in labelled plastic containers filled with water (see Plate 4.12). It was envisaged that cross contamination is likely; hence, each particular mix composition was soaked separately. Also to ensure a thorough soaking, a wire mesh was placed in the bottom to a height of not more than 50mm from the bottom. The samples were monitored from time fractions of an hour to a period of ten days within which the samples were weighed using a weighing balance capable of weighing to within ±2g, and the values recorded. At the end of the ten days soaking regime while monitoring the amount of water that these samples are capable of holding up in their pores, a UCS test was conducted on them. The series of weighings were plotted in graphs to show the percentages of water absorption with time while the strength values obtained from the UCS test were used to assess the strength reduction indices of the different samples, as a percentage of the samples of the same age but not subjected to the extreme conditions of soaking in water.

Plate 4.12 Test samples soaked to monitor water absorption

Chapter Five: Results

5.1 Consistency Limits Tests

Table 5.1 shows the results of the Liquid Limit test carried out on Lower Oxford Clay (LOC) soil in its natural un-stabilized state. It has been established that water content is critical to the physical appearance of soil fabric and regulates the chemical properties of a soil. Water being a universal solvent, its presence in soil determines to a large extent the possible reactions the minerals in the soil, or added to the soil, can undergo. From Table 5.1, at a moisture content of 28.9% a penetration of about 14.8mm was achieved. On further addition of moisture the inter-particle friction was reduced resulting in increased penetration of the test needle. This trend continued and if moisture addition is not restricted, at a certain point the solid soil fabric would turn to liquid and flow.

It has been well established by other researchers that the liquid limit of LOC increases upon the addition of a stabilizer (Lime) (Bell, 1996). This increase in Liquid Limit has also been ascertained not to exceed the lime fixation point which is the point where further addition of lime creates no observable effect on the physical and index properties of the stabilized soil. The current research considers the investigation of liquid limit variation with lime addition to be common place knowledge. Therefore more effort was aimed at the determination of the engineering properties of the novel material formulations upon blending with lime/GGBS.

The properties this research aims to explore are those resulting from the activation of GGBS with lime. The activation of GGBS with lime is relatively time dependent. Hence, conducting a liquid limit test on LOC-Lime-GGBS system will be a repetition of the liquid limit on LOC-Lime system which produces a rather more instantaneous effect on the index properties and has been very well researched by other researchers such as Bari Thomas (Bari, 2001). Though the effects of lime on the consistency limits of Lower Oxford Clay are instantaneous, further research has proved that a period of mellowing changes the results though on a very little scale (Bari, 2001).

Fig 5.1 is the representation of the determination of the Liquid Limit (LL) of Lower Oxford Clay soil in its natural un-stabilized state. A linear relationship is seen to exist between moisture content addition and penetration. At low moisture content the penetration was very low while at high moisture contents proportionately high penetration was observed. The liquid limit is the point that corresponds to a penetration of 20 mm. This corresponding moisture content is approximately 33.2%. On investigating the Plasticity Limit (PL), an average of three results was taken. The moisture content of the three different trials for PL was determined after a 24-hour period of drying in the oven set at 105°C and this averaged at 19.7%. The Plasticity Index value is normally taken as the difference between the Liquid Limit and the Plastic Limit and this was determined to be 13.5. Thus, the Consistency Limits of the Lower Oxford Clay used were found to be approximately:

LL = 33

PL = 20

PI = 13

Table 5.1 Moisture Content Vs Penetration of the Lower Oxford Clay used.

Moisture Content (m/c) (%)	Penetration (mm)
28.9	14.8
31.4	17.3
33.4	20.3
34.7	22.4

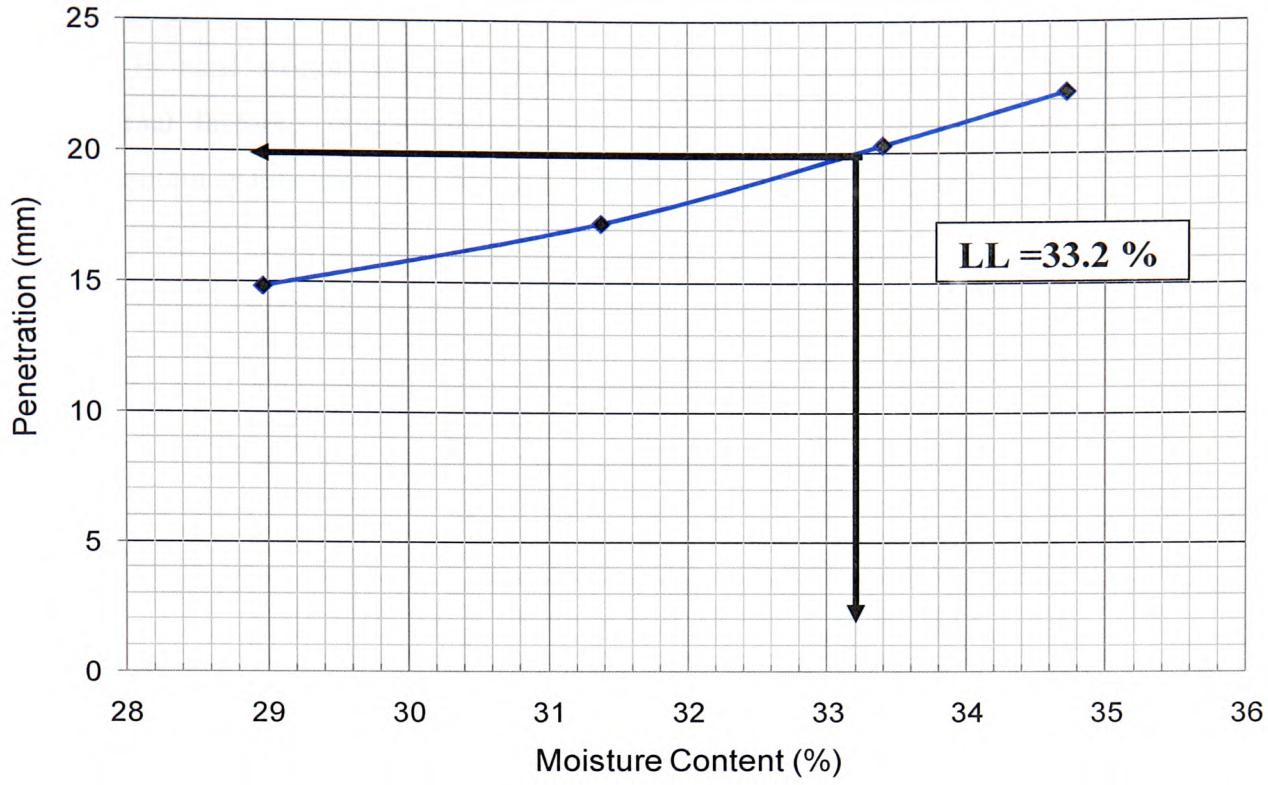


Fig. 5.1 Determination of Liquid Limit of Lower Oxford Clay used.

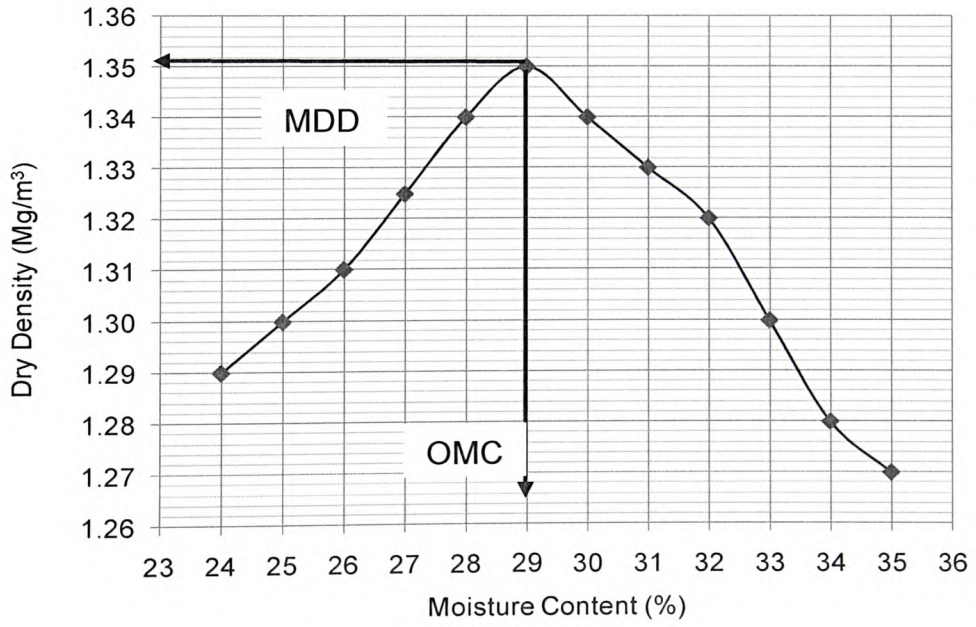
5.2 Proctor Compaction Test

The BS Proctor compaction test was carried out according to the British Standard Specification BS 1377: Part 4: 1990 which is the determination of the Dry Density-Moisture Content relationship using either a 2.5kg or 4.5kg rammer methods. Table 5.2 shows the dry density-moisture content relationship of Lower Oxford Clay at its natural un-stabilized state, while Figure 5.2 illustrates the determination of Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) of the Clay soil. From the plot, the dry density is seen to increase with increase in moisture content. This increase continues until a peak dry density value was reached at about 1.35 Mg/m^3. This peak is the Maximum Dry Density (MDD) and it occurs at a given moisture content, known as the Optimum Moisture Content (OMC). This corresponded to a moisture content of 29% on the moisture content axis. Better strength and compaction has been observed to occur from compacting soil samples at or near the OMC, which yields the maximum dry density. In practice, environmental conditions may pose challenges which may affect the achievement of compaction at OMC. These challenges which include the effects of evaporation, especially

in tropical climates, could compromise the attainment of MDD. To counteract these effects especially in road works where large areas of soil are exposed to the influence of environmental factors, compaction of soil is targeted at slightly above the optimum moisture content as this takes into account evaporation and other moisture loses.

Table 5.2 Moisture content/Dry Density relationship

Moisture Content (%)	Dry Density (Mg/m^3)
24	1.29
25	1.30
26	1.32
27	1.33
28	1.34
29	1.35
30	1.34
31	1.33
32	1.32
33	1.30
34	1.28
35	1.27



MDD = 1.35 Mg/m^3; OMC = 29%

Fig. 5.2 Determination of Maximum Dry Density and Optimum Moisture Content of Lower Oxford Clay used in the current work.

5.3 Compacted Material Properties

The previous two sections dealt with the general engineering properties of the un-stabilised material (LOC). This section and the subsequent sections show the results of the different experiments carried out to investigate the properties of the novel stabilised material formulations. These properties are showed in two parts, the first part (Section 5.3) assessing the Unconfined Compressive Strength (UCS) of the compacted material, the volume stability during moist curing, and the compressibility of the stabilized moist cured material upon loading. Then the second part (Section 5.4) sets to investigate the hydric properties which includes water absorption, permeability and the effect of these two parameters on other properties such as the soaked UCS, the Strength Reduction Index (SRI), the volume stability upon soaking and the compressibility of the material mixture after soaking in water.

5.3.1 Unconfined Compressive Strength

Figure 5.3 shows the development of Unconfined Compressive Strength (UCS) of Lower Oxford Clay (LOC) stabilized with varying percentages of various stabilizers at a compaction moisture content of 23% and cured in a humid environment for a period of up to 28 days (refer to Section 4.4). Up to 16% of the dry weight of soil was used as the maximum amount of stabilizer addition. For lime used on its own, (Fig. 5.3(a)), the dosage was applied in steps of 4% starting from 0%, defining five levels of stabilization (0%, 4%, 8%, 12% and 16%). The samples with no stabilizer additions (0%) were taken as baseline or controls. The results presented in Fig. 5.3(a) show a trend of increasing strength development with increase in the amount of stabilizer utilized (Lime). When no stabilizer was added to the soil there was no strength development observed. However, on the addition of 4% stabilizer the strength development increased rapidly. On further stabilizer addition to 8% the strength development ceased beyond this dosage, and the rate of strength increase started falling especially at 7 and 14 days of curing. This fall in the rate of strength increase continued as the amount of stabilizer was increased further to 12% and finally to 16%.

If the strength is plotted against amount of stabilizer utilized an optimum strength will be observed at 4% stabilizer addition see Fig. 5.3(b). Another trend that was observed is an increase in strength development with age of curing. Each of the various levels showed a positive trend of strength development from the 7-day test through to the 28-day strength determination. The specimens stabilized at 4% level of stabilizer addition started with a 7-days UCS value of 520kN/m^2 and increased progressively throughout the observation period to attain a value of above 700kN/m^2 after 28 days moist curing. This represented a 38% increase in strength development from the 7-days strength. The 8% stabilizer addition showed a high strength development even though this represented a decrease when compared with that of 4%. This implies a drop in strength of the lime system after the attainment of the optimum strength, probably between 4%-6% lime dosage. Observing the strength development with respect to curing time, the results show that the 12% and 16% stabilizer additions recorded 12% and 10% rate of strength development respectively from the 7-days to the 28-days UCS results.

Fig. 5.3(C) shows the strength development pathways of LOC stabilized with blended compositions of Lime-GGBS at 23% compaction moisture content. A maximum of 16% stabilizer level was the target level of stabilizer addition. To achieve this, five stabilization levels were defined which combined the stabilizers thus: 16%Lime-0%GGBS, 12%Lime-4%GGBS, 8%Lime-8%GGBS, 4%Lime-12%GGBS and 0%Lime-16%GGBS. From the results, it is obvious that the specimens with 4% Lime-12%GGBS performed best in terms of strength development. The strength of these samples increased from 800kN/m^2 at 7-days of testing to a strength value of 1200kN/m^2 after 28-days, representing a growth rate increase of 50%. All the other blended mixes, though with lower strength values, also showed the trend of increasing strength development with incremental replacement of lime with GGBS to achieve an optimal blending ratio of 1:3. All the blended stabilisers showed increased strength with extended curing ages. The GGBS component of the blend is believed to have enhanced the strength from the maximum strength of 720kN/m^2 observed in the lime system (Fig. 5.3(c)) to 1200kN/m^2 in the lime-GGBS system (Fig. 5.4) which is a 67% rate of strength increase. Specimens without any lime to activate the GGBS (0%lime-16%GGBS) achieved comparable strength magnitude to those where lime was not replaced by GGBS (16%Lime-0%GGBS).

From the foregoing, it is believed that both lime and Lime-GGBS are best suited at a particular point of addition or blending ratio (optimum). Any further stabilizer additions or change in blending in excess of this point did not seem to contribute to further increase in strength development. This optimum stabilizer dosage was between 4-8% Lime for the lime system and a blending ratio of 1:3 in 4%Lime-12%GGBS for the Lime-GGBS system.

Fig. 5.4(a) shows the strength development of LOC stabilized with varying dosages of Portland cement (PC) (0, 4, 8, 12 and 16%), at compaction moisture content level of 23%. The graph shows a trend of increasing compressive strength with increasing PC addition. It also shows a trend of increasing strength development with extended age of curing. This is clearly pointing to the fact that as more soil is being replaced by PC, the more the strength development of the composite material increases. PC is known to bind most materials, although its use may not be justified if the whole-life-cycle costing is a prime consideration. This is as a result of the economic and environmental strain the manufacture of PC presents. From Figure 5.4(a) and (b), each 4% increase in stabilizer content increased the bearing capacity of the stabilized material proportionately. The 4% PC specimen achieved a 7-day Unconfined Compressive Strength (UCS) value of about 240 kN/m^2 and after 28 days of moist curing the strength increased to 400 kN/m^2, representing a strength development rate of increase of 67%. Similarly, the 8, 12 and 16% levels of PC addition increased strength by 24%, 39% and 46% respectively, relative to the respective 7-day strength it is clear that more stabilizer addition will represent a corresponding increase in strength development. However, this is not cost effective and negates current efforts aimed at arresting global carbon emission, natural resource utilization/conservation and energy reduction apprehensions occasioned by the manufacture of PC on a global scale.

When PC was replaced by up to 75% with GGBS as illustrated by 4%PC-12%GGBS in Fig.5.4(c), the implication of cost reduction of utilizing marginal and/or by-product materials such as (GGBS) in soil stabilization becomes more obvious. This is evident from

the fact that 75% PC replacement could translate to enormous financial savings accruing from reduced material utilization, energy and cost of production of only 25% PC. It is also clear from Figs. 5.3(c) and 5.4(c) that at a given stabilizer blend (4%Lime-12%GGBS, 4%PC-12%GGBS), the best performance of the stabilized material is obtained. After this optimum stabilization level is reached, strength starts falling when lime or PC is substituted further with GGBS. It is possible that with tactical manipulation of such variables as density, moisture content and compaction energy, higher strengths are achievable. However, for ease of comparison and data analyses the volume and compaction energy parameters were kept constant while the density and moisture contents were varied.

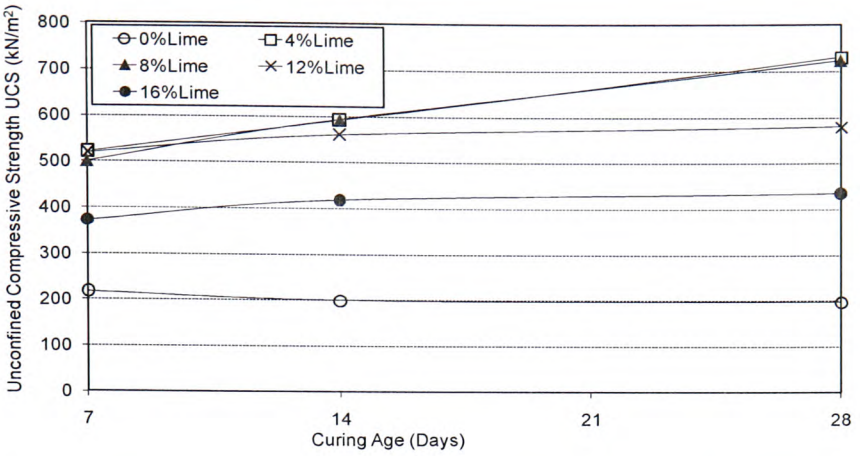
Figure 5.5 - 5.7 show the effect of moisture content variations in different blended mixes. Four compaction moisture content levels of 23, 28, 33 and 38% of the dry weight of stabilized materials were defined (see appendix A). From the compaction test carried out on the LOC sample, a moisture content of 29% was established as the optimum compaction moisture content. However, soil stabilization with lime (CaO) is exothermic in nature, requiring much water due to the heat liberated during reaction. To accommodate any water demands by the system, water was added at increments of 5% from the 23% to create a large window of moisture variation. Having in focus a desire to achieve specimen stabilization that is wet of the optimum, the moisture addition was extended to cover up to 38%. From Fig. 5.5a which shows LOC stabilized at 23% compaction moisture content with the different blended compositions of Lime-GGBS. The two stabilization extremes of 16%Lime-0%GGBS and 0%Lime-16%GGBS showed low strength development. However, on the replacement of lime with GGBS the strength development started increasing, further replacement to achieve 8%Lime-8%GGBS showed increase in strength with a maximum increase occurring at 4%Lime-12%GGBS (1:3 ratio) level of replacement. Another trend that was observed was that of increasing strength development with extended curing age. Lime has the tendency to carbonate upon exposure to air, after the lime fixation point is reached; any excess un-reacted lime is supposedly believed to facilitate carbonation reaction upon exposure to air. The effect of carbonation on strength development is not well known but it is believed to aid loss in strength. This explains why the decline in strength development with the addition of more lime was possible. Fig.

5.5(b) shows LOC stabilized at 28% moisture content with varying blends of Lime-GGBS. This was stabilized at 16% maximum stabilizer level; the replacement of lime with GGBS improved the 7-days strength of 12%Lime-4%GGBS by 75%, 8%Lime-8%GGBS by 100% and 4%Lime-12%GGBS by 138%. However, with the effect of curing time, 500 kN/m^2 which is the 28-days UCS value for 16%Lime-0%GGBS blended mix was enhanced with the addition of GGBS to 1100 kN/m^2 for 12%Lime-4%GGBS, 1380 kN/m^2 for 8%Lime-8%GGBS and 1480 kN/m^2 for further lime replacement up to 75% (4%Lime-12%GGBS). Fig. 5.5c and 5.5d are the strength development of stabilized LOC comparing different blended mix compositions at moisture content of 33 and 38% respectively. From Fig. 5.5c the trend is similar to those of Fig 5.5b, however, higher strength values were obtain as shown by 4%Lime-12%GGBS with 28-days strength of above 1600 kN/m^2 against 1500 kN/m^2 in Fig. 5.5b. In Fig. 5.5d the effect of higher moisture translated to a reduction in the Unconfined Compressive Strength value with the 4%Lime-12%GGBS specimen showing a 14% strength reduction relative to that of 33% moisture content in Fig. 5.5c. From the foregone, it could be extrapolated that optimum moisture content for the formulations fell within the 33% compaction moisture content and the rate of strength development was higher within the first 7 days of curing, this is evident from the sharp gradients of the 7 to 14 days points in the graphs.

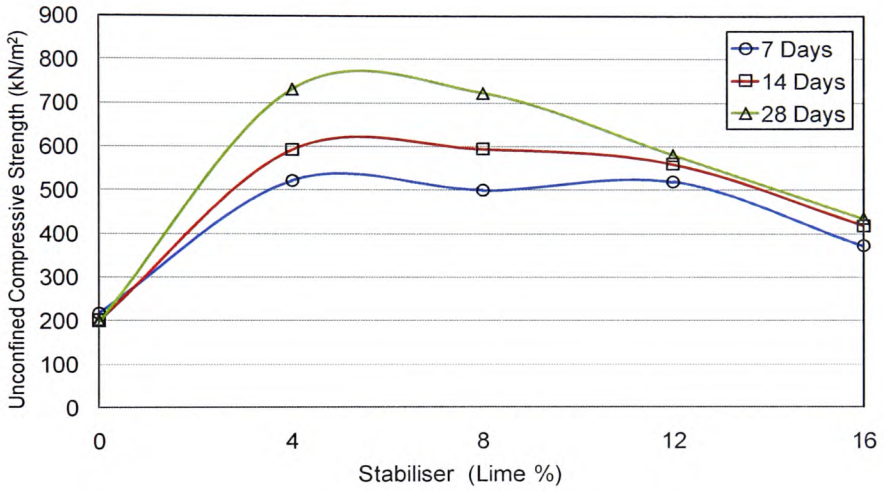
From Fig. 5.6 and 5.7 the effect of moisture content variation on the different blended compositions was investigated. Form Fig. 5.6a it is observed that strength development increased with the replacement of lime in the Lime-GGBS systems. This increase continued till an optimum was reached at 4%Lime-12%GGBS (1:3 replacement ratio). Similarly, the strength development also increased with increased curing time. This was observed to be a general trend with all the blended compositions. Another trend that was observed was that of increasing with increased compaction moisture content until an optimum value was reached. The difference in moisture content and variation in mix composition are also responsible for the variation in strength development within each blended mix composition. This is possible as the composition of different compounds within a given mix will vary and will affect the mix differently to other mixes.

Fig. 5.8 and 5.9 presents the results obtained by increasing specimen compaction density from 1.79 Mg/m^3 to 2.04 Mg/m^3 at varying compaction moisture contents. The best two performing mix compositions were investigated further using cylindrical moulds of the same dimensions. More matter was packed into the specimens to increase the specimen densities. Increasing the specimen density from 1.79 Mg/m^3 to 2.04 Mg/m^3 saw the compressive strength of the different mixes doubled as shown in Fig. 5.8 and 5.9. From Fig. 5.8a, b and c a maximum compressive strength value of about 2500 kN/m^2 was observed at compaction moisture content of 28% and curing age of 28 days. It is clear from the results that at 28% compaction moisture content, strength development was higher. The replacement of lime with GGBS reduced the moisture requirement of the Lime-GGBS system. This was possibly through a reduction in the intensity of lime as more lime was been replaced. The samples showed better strength upon long time curing; it is believed to be as a result of the chemistry that is going on within the system which may require time to complete. High compressive strength development is an indication of a likely durable material, it was considered necessary to carry out different durability tests to check the durability parameter of the novel material mixtures and possibly enhance material robustness.

(a)



(b)



(c)

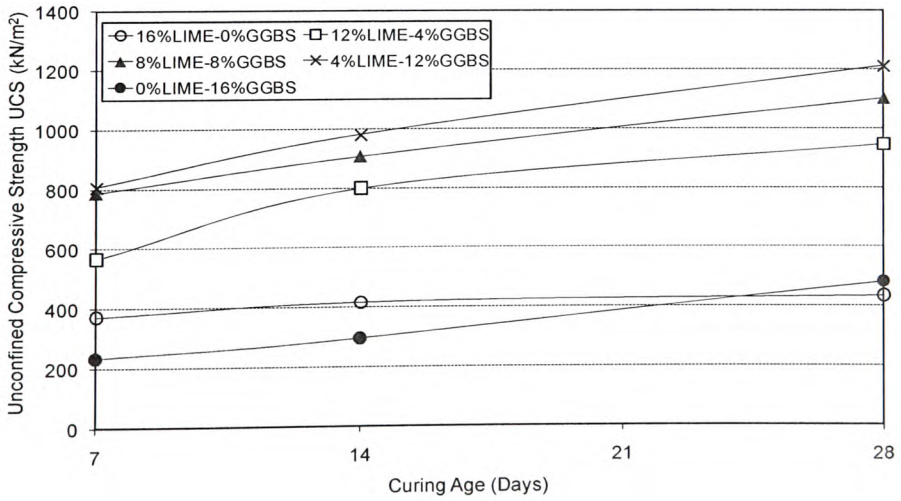


Fig. 5.3 Unconfined Compressive Strength (UCS) of stabilized Lower Oxford Clay for varying stabilizer dosages, at 23% moisture content (a) Lime Stabilizer (b) Strength Vs. stabilizer content, and (c) Lime-GGBS blended stabilizer

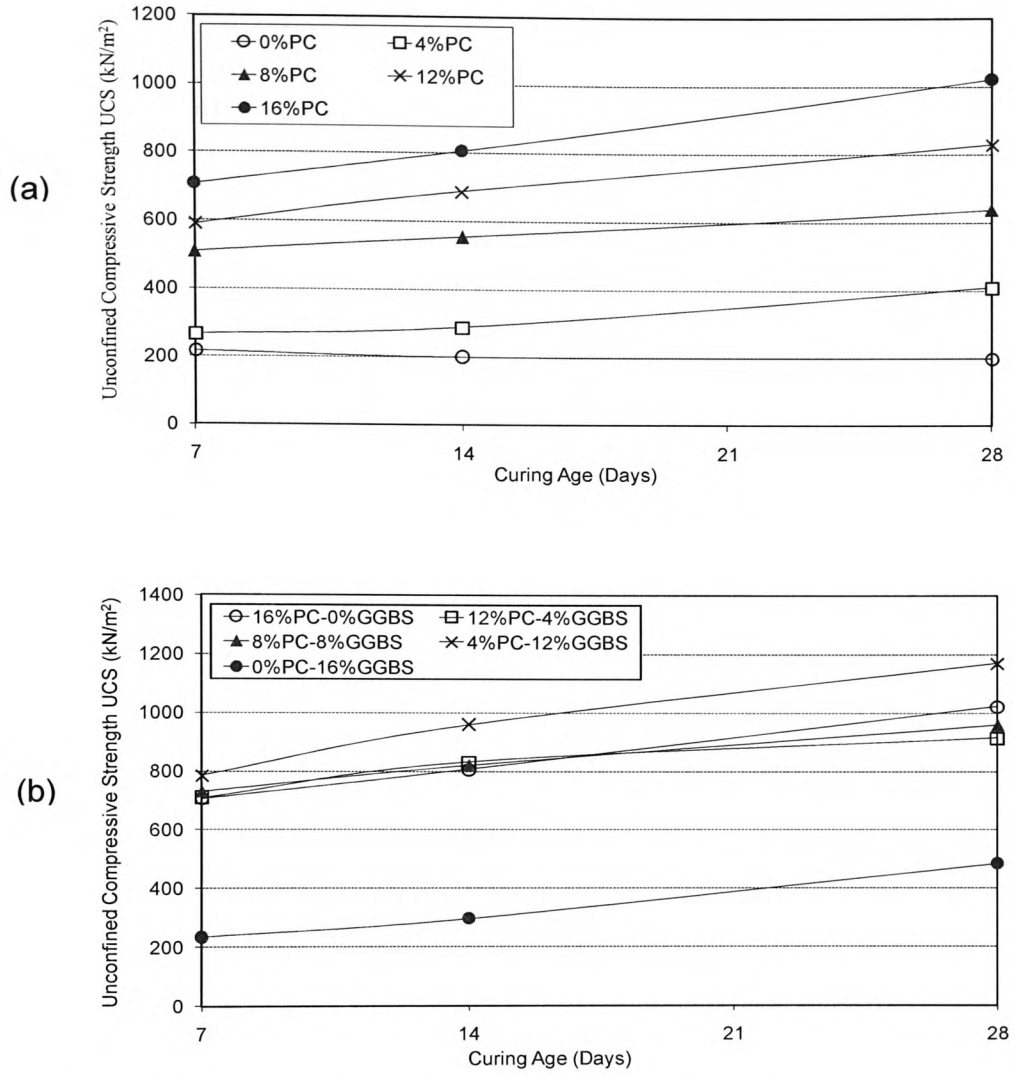
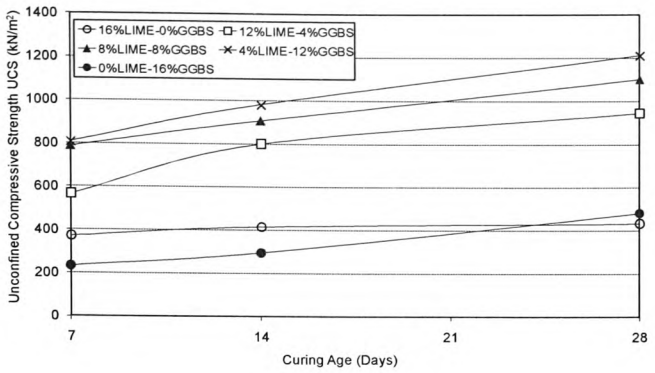
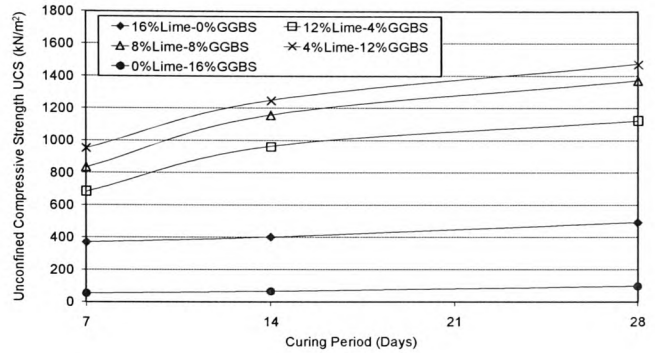


Fig. 5.4 Unconfined Compressive Strength (UCS) of stabilized Lower Oxford Clay for varying stabilizer dosages, at 23% moisture content. (a) Portland cement (PC) stabilizer, and (b) PC-GGBS stabilizer

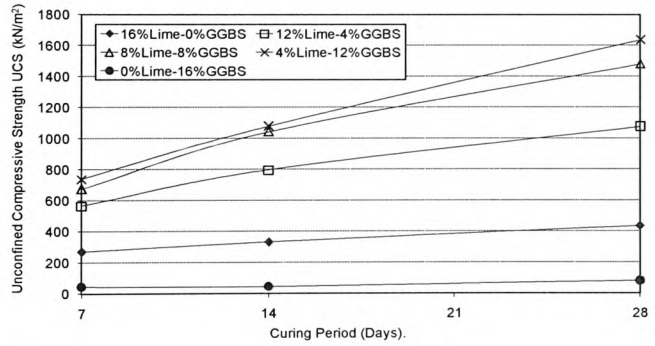
(a)



(b)



(c)



(d)

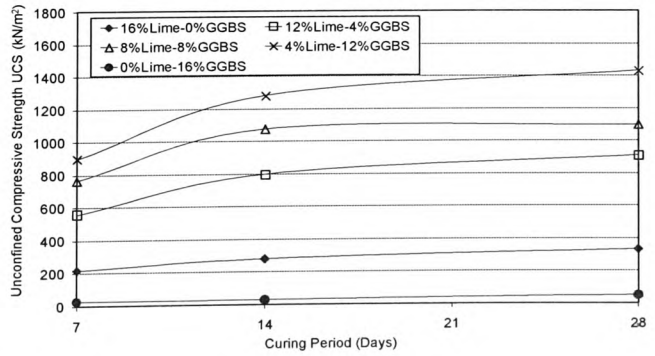
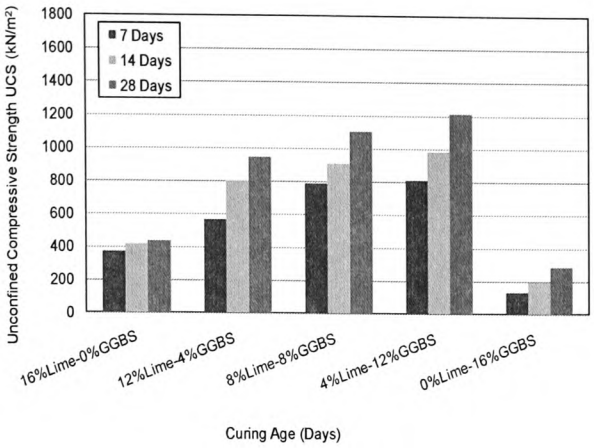
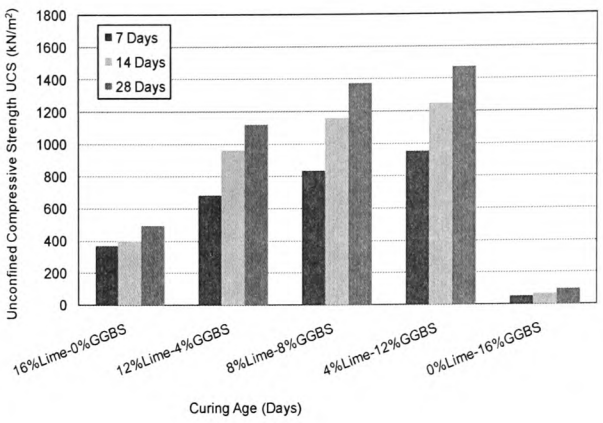


Fig. 5.5 Unconfined Compressive Strength of Lower Oxford Clay stabilized with lime-GGBS at varying moisture contents. (a) 23% moisture content, (b) 28% moisture content, (c) 33% moisture content, and (d) 38% moisture content.

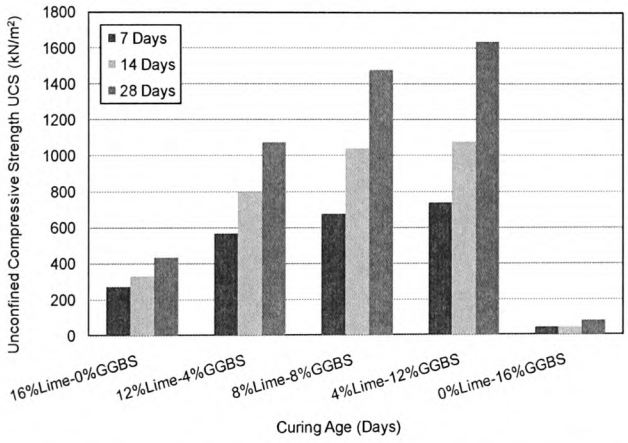
(a) – 23% mc



(b) – 28% mc



(c) – 33% mc



(d) – 38% mc

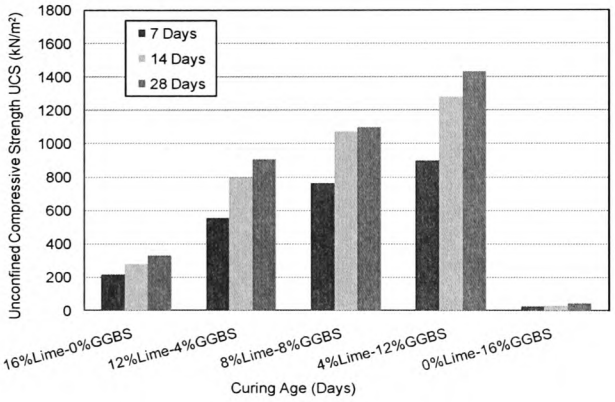
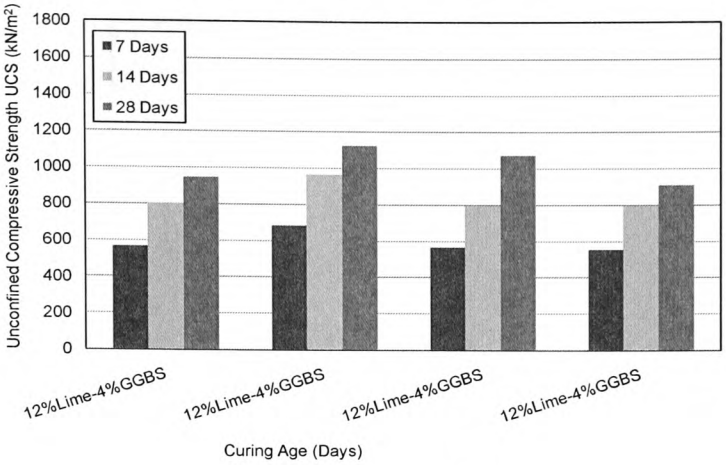
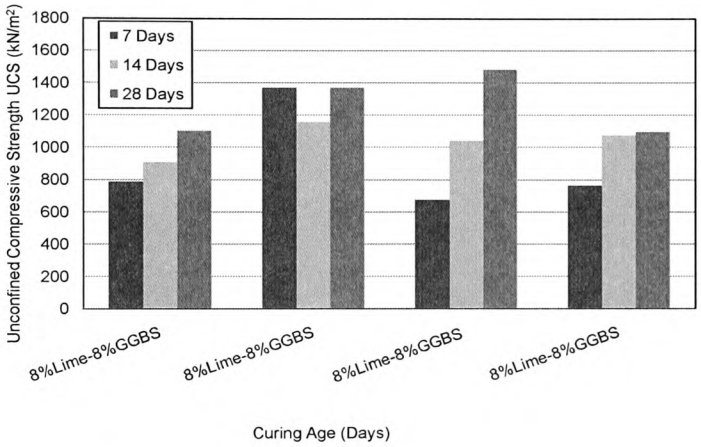


Fig. 5.6 Effect of Compaction Moisture Content on the Unconfined Compressive Strength of Lower Oxford Clay stabilized. (a) 23% moisture content, (b) 28% moisture content, (c) 33% moisture content and (d) 38% moisture content.

(a) – 12%Lime-4%GGBS



(b) – 8%Lime-8%GGBS



(c) – 4%Lime -12%GGBS

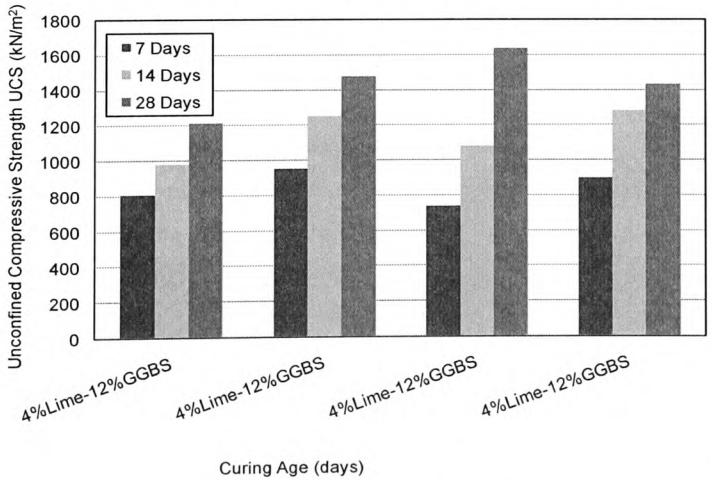


Fig. 5.7 Unconfined Compressive Strength (UCS) of Lower Oxford Clay stabilised with Lime-GGBS blends effect of decreasing lime content (and increasing GGBS content), each blend shown at varying compaction moisture contents.

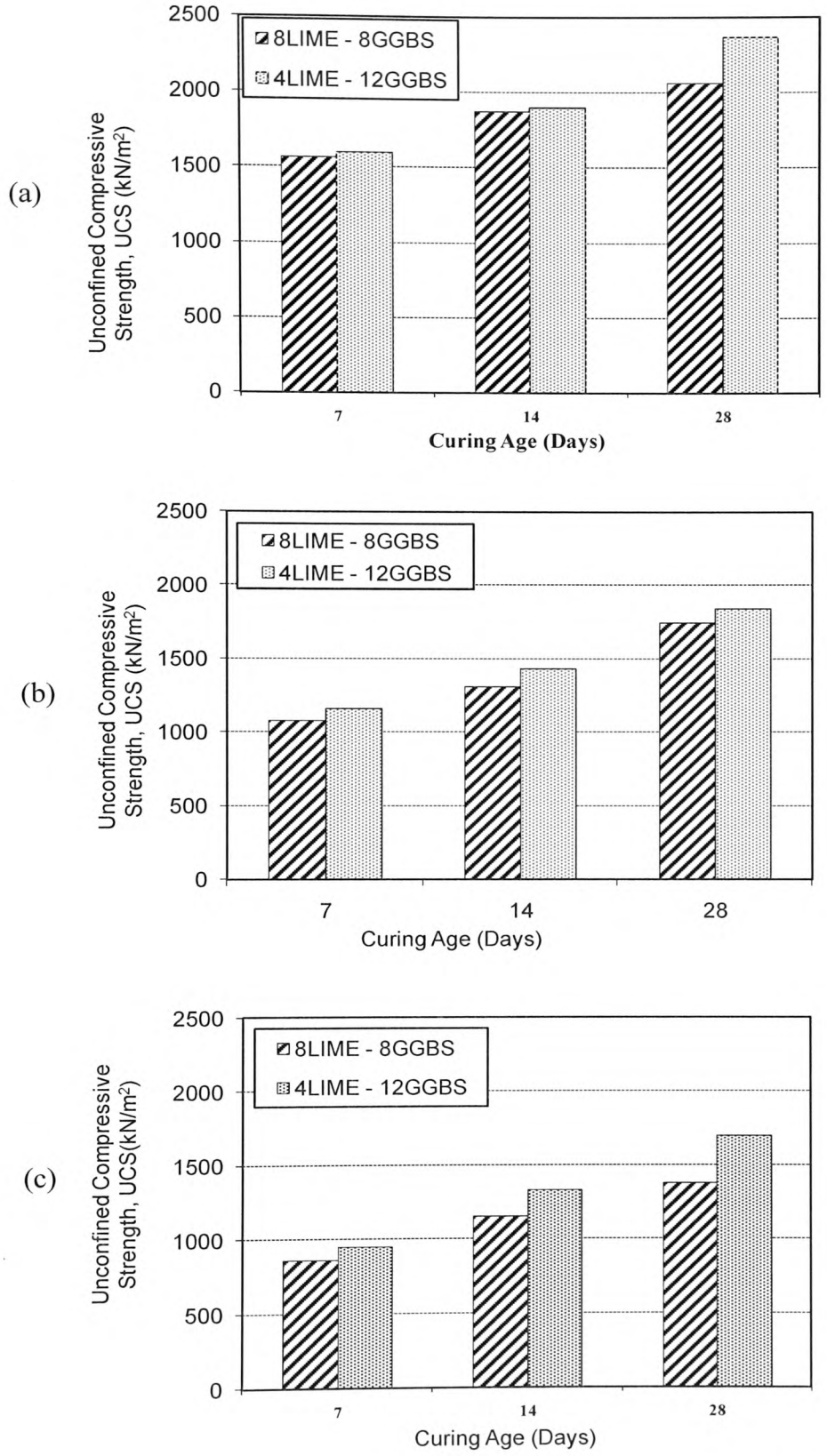


Fig. 5.8. Effect of density on the UCS of Lower Oxford Clay stabilized with varying proportions of Lime-GGBS blended mixes at varying moisture contents. (a) 28% moisture content, (b) 33% moisture content, (c) 38% moisture content.

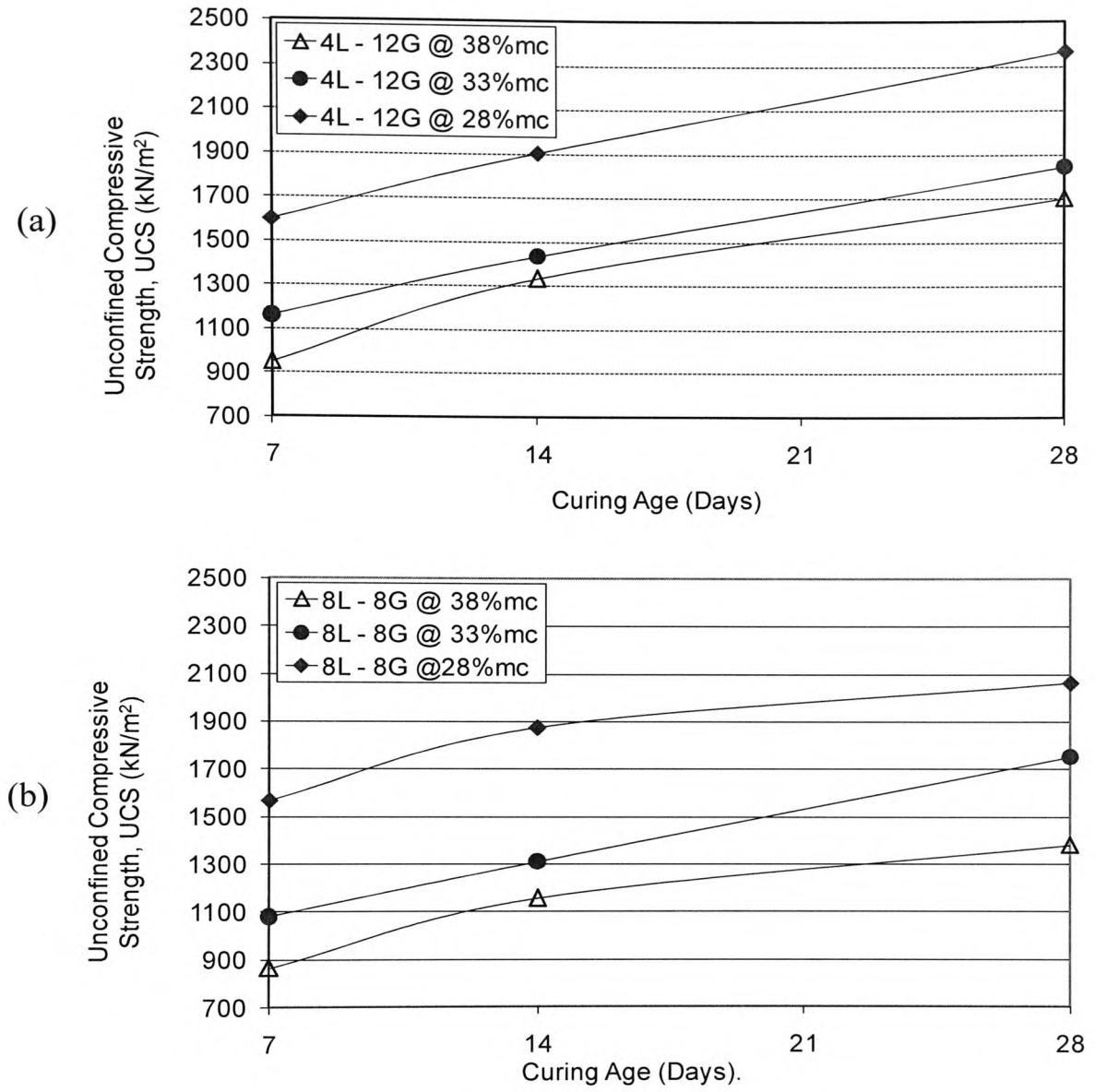


Fig. 5.9. Effect of increasing compaction density on the UCS of Lower Oxford Clay stabilized with Lime-GGBS blended stabilizer at different moisture contents. (a) 4%Lime-12%GGBS, and (b) 8%Lime-8%GGBS. L = Lime, G = GGBS, and mc = Moisture Content.

5.3.2 Compressibility/ Deformation

Table 5.2 shows the effect of loading on the stabilized materials in terms of reduction in the overall length of the test material. There is no strong trend in the variation in sample deformation upon curing beyond 7-days. Overall, the magnitudes of the deformation are all very low, being at most at 2% of the original sample length (2% of 100mm = 2mm). A deformation of only 2mm for a 100mm layer is considered insignificant. Overall, the 16%Lime-0%GGBS stabilizer showed more compressibility potential compared to the lime-GGBS stabilizers where compressibility was minimal and negligible even though more bearing load was recorded on these test specimens. For the 16%Lime-0%GGBS to fail at lower load applications with higher compressibility, the results suggest that the replacement of lime with GGBS offered enhanced compaction benefits resulting from reduced void spaces. From the Table, for about half of the test specimens, it was observed that at early age of curing the tendency to compress was higher and this tendency reduced as the curing ages increased. The reduction continued till after 56 days when there appeared to be an increase again in compressibility. At higher compaction moisture contents, the compressibility was generally lower though in terms of the load carrying capacity of the stabilized materials, lower strength values were observed. Perhaps the apparently lower Compressibility is due to the fact that the monitoring of deformation stopped as soon as the test samples stopped carrying any load.

Table 5.3 Deformation of test samples during testing for UCS after different curing periods, as a percentage of the original length of samples.

S/N	Sample	7 days Ave Extension (%)	14 days Ave Extension (%)	28 days Ave Extension (%)	56 days Ave Extension (%)	90 days Ave Extension (%)	150 days Ave Extension (%)
1	4L-12G-23%mc	1.5	1.5	1.5	1.5	2	2
2	4L-12G-28%mc	1	1	1	1	1	1.5
3	4L-12G-33%mc	1	1	1	1	1	1

S/N	Sample	7 days Ave Extension (%)	14 days Ave Extension (%)	28 days Ave Extension (%)	56 days Ave Extension (%)	90 days Ave Extension (%)	150 days Ave Extension (%)
4	8L-8G-23%mc	2	1.5	1.5	1.5	1.5	2
5	8L-8G-28%mc	1.5	1	1	1	1.5	1.5
6	8L-8G-33%mc	1	1	1	1	1	1

S/N	Sample	7 days Ave Extension (%)	14 days Ave Extension (%)	28 days Ave Extension (%)	56 days Ave Extension (%)	90 days Ave Extension (%)	150 days Ave Extension (%)
7	16L-0G-23%mc	2	2	1	1	1.5	2
8	16L-0G-28%mc	2	1.5	1.5	1.5	1.5	1.5
9	16L-0G-33%mc	1.5	1.5	1	1	1	1.5

5.3.3 Volume Stability upon Wetting

In assessing the volume stability of the novel material formulations, two assessment criteria were followed – stability within the moist curing period of 7 days (dry) and that within the soaking period of up to 50 days (wet), that was carried out immediately after the 7-day moist-curing period Fig. 5.10 shows the result of the linear expansion measurement of stabilized LOC within the moist curing period of 7 days. Fig. 5.10(a) shows the linear expansion when LOC was stabilised with a high percentage of lime, of up to 16%, at varying compaction moisture contents. From the results, the general trend is that of reducing expansion with increased compaction moisture content. At all the compaction moisture contents, the LOC stabilised with 16% lime showed high rate of increase within the moist curing period, reaching a maximum value of above 4.5% within 7 days of expansion measurement. The test specimens made at a 28% compaction moisture content expanded more than all the others, followed by those at 23%. Figure 5.10(b) show the expansion at the end of the moist curing period. The Figure shows peak expansion at 28% compaction moisture content, with lower expansion magnitudes for test specimens compacted drier or wetter of this optimal value.

Upon the replacement of lime with about 50% GGBS to achieve the blended composition of 8%Lime-8%GGBS (see Fig. 5.11), the linear expansion behaviour of the stabilized material took a different shape. The gradient of the plots suggest that the material rapidly resisted continued expansion, compared with the linear gradients of Fig. 5.10a. However, the general trend was still that of reduced expansion with increase in the compaction moisture content. Compared with the results in Fig. 5.10a at each of the compaction moisture contents, higher expansion at 7-day were recorded with the lime stabilisation than with the lime partially replaced with GGBS. Test specimens compacted at 28% m/c continued to show maximum linear expansion (see Fig.5.11(b)). With further replacement of lime to attain the 4%Lime-12%GGBS which represented a 1:3 lime to GGBS ratio or 75% lime replacement, there was a marked effect on the linear expansion. All the test samples at the different compaction moisture contents expanded sharply within the first three days, beyond which no further expansion was observed. There is a trend of reduced expansion with increasing compaction moisture content, with test specimens compacted at 28% still showing maximum expansion (Fig 5.12(b)).

From all the results on linear expansion, it is clear that expansion reduced with lime replacement as much as with moisture content variation and increasing moist curing period.

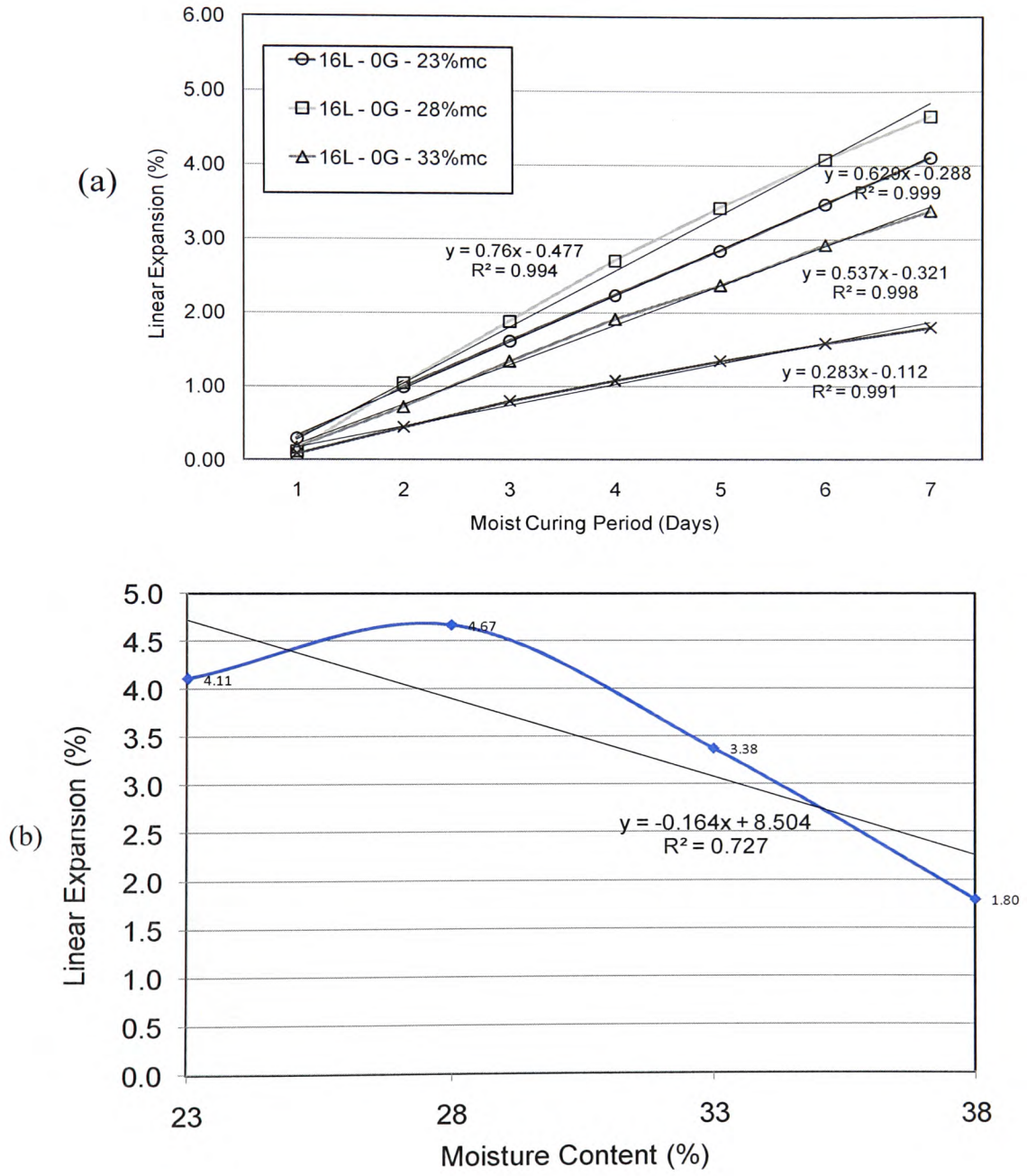


Fig. 5.10 Linear Expansion measurements of stabilized LOC within the moist curing period (a) 16%Lime-0%GGBS (b) Expansion at the end of 7-day moist curing period.

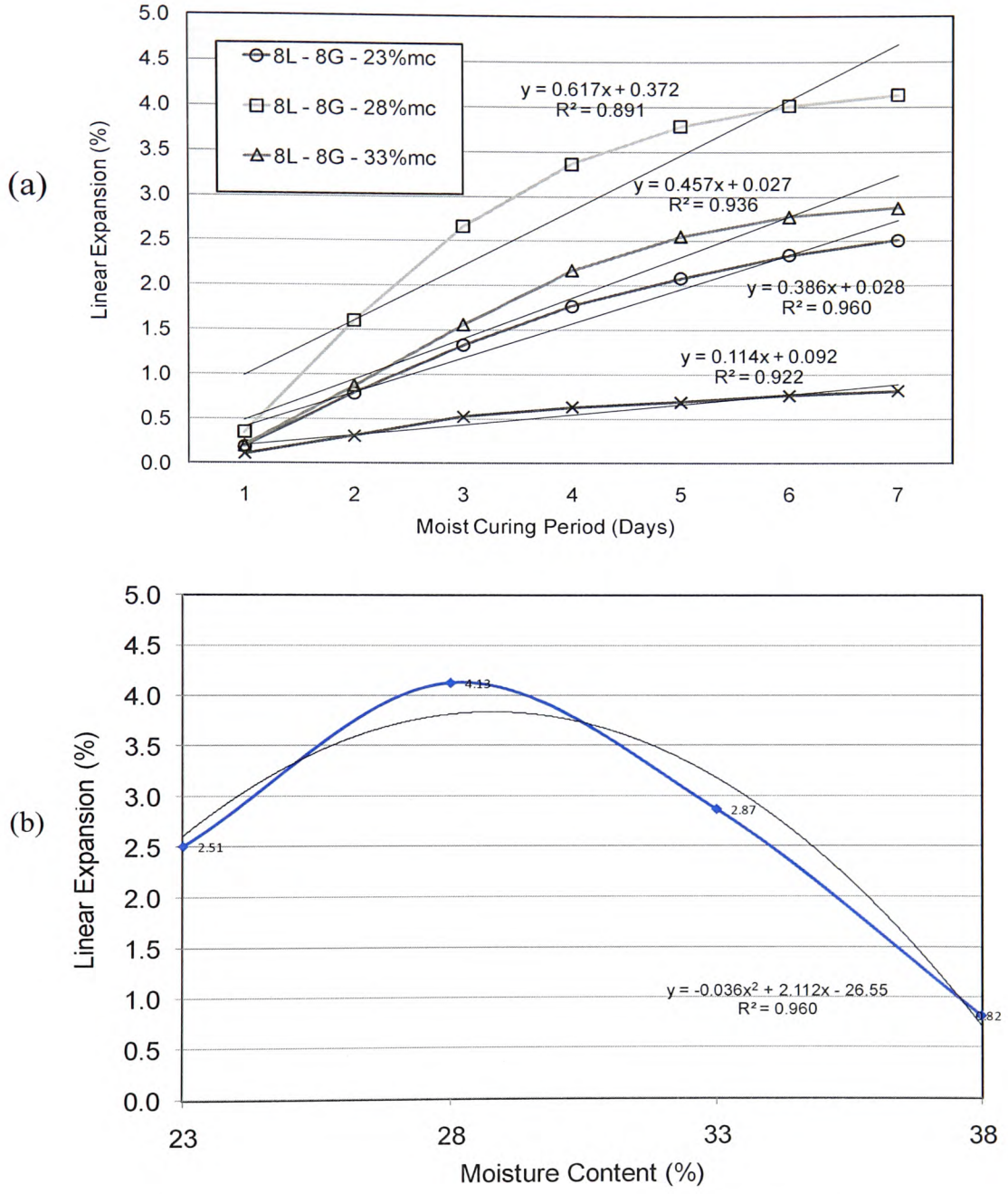
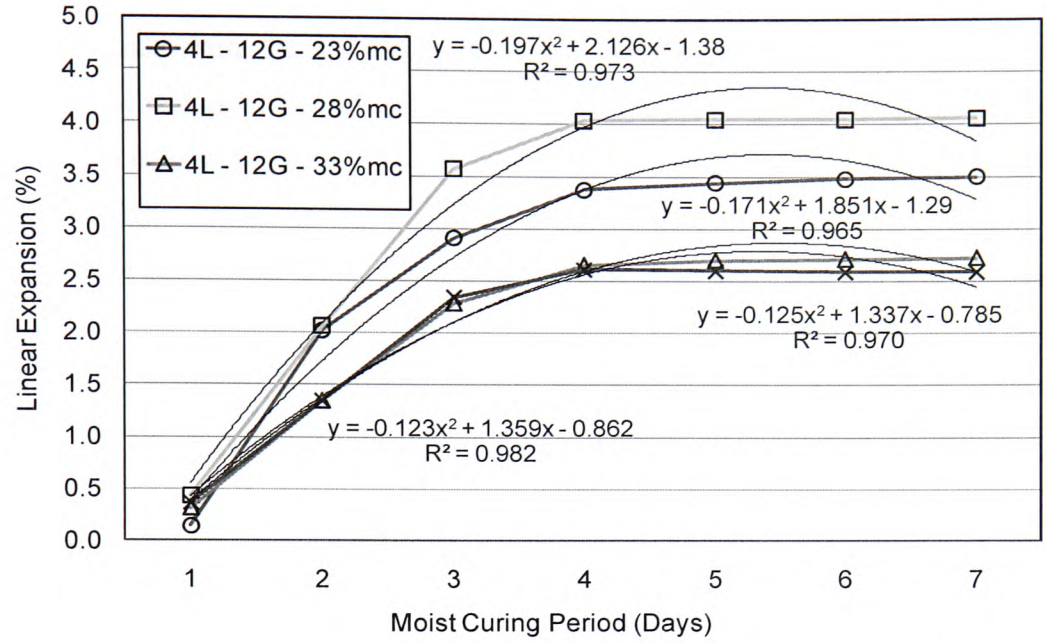


Fig. 5.11 Linear Expansion measurements of stabilized LOC within the moist curing period (a) 8%Lime-8%GGBS (b) Expansion at the end of 7-day moist curing period.

(a)



(b)

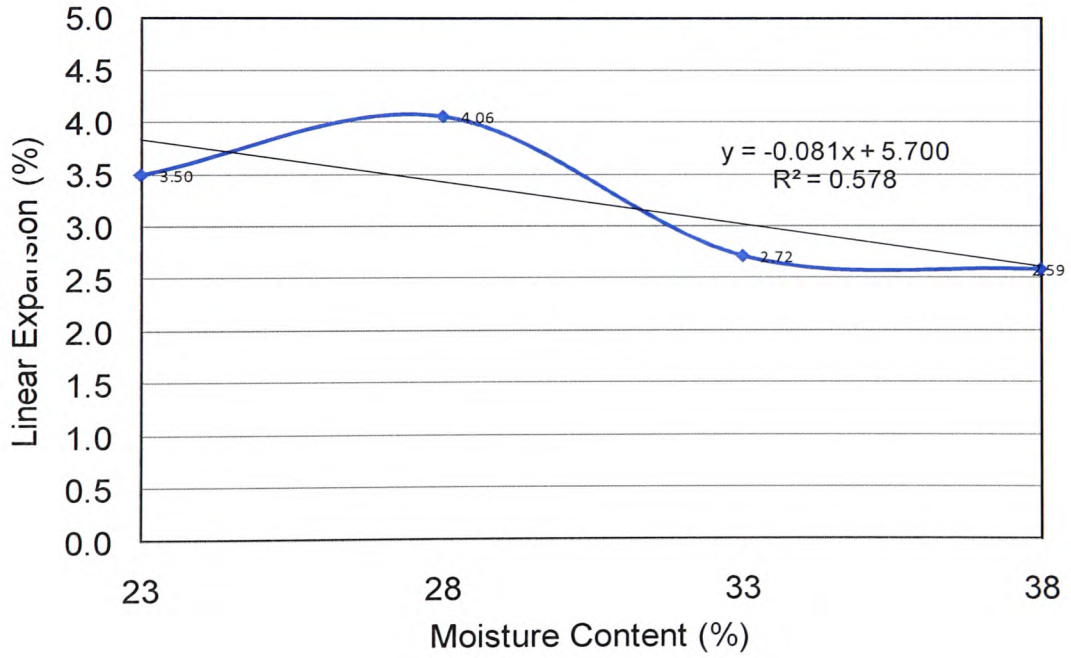


Fig. 5.12 Linear Expansion measurements of stabilized LOC within the moist curing period (a) 4%Lime-12%GGBS (b) Expansion at the end of 7-day moist curing period.

5.4 Durability of the stabilised material

This section deals with the properties and effects of subjecting the stabilised material formulations to the damaging influence of water. When water gains access to stabilised soil materials, there is usually some form of wetting and continued chemical reaction that takes place. This might either facilitate continued strength development, or result in the reduction of the prevailing bearing capacity of the material. Different experiments were therefore performed, as reported in this section, to investigate the effect of water (flooding) on the stabilized materials used in the current research.

5.4.1 Water Absorption

The water absorption of Lime-GGBS stabilized Lower Oxford Clay was investigated and the results are presented in Figures 5.13-5.27. Fig. 5.13 shows the water absorption profile of Lower Oxford Clay stabilized Lime-GGBS blended stabilizers after moist curing for 7 days, during 10 days of complete submersion in water. From Fig. 5.13(a) which shows the stabilization of Lower Oxford Clay with high lime content at 16%Lime-0%GGBS at varying compaction moisture contents, the water absorption was observed to be significant, although lower than 20%, quoted for most stabilised clay materials (Tasong et al, 1999). This set of specimens was used as control, and therefore there was no replacement of lime. From the results, there are two identifiable trends. Firstly, one of sharp increase in the rate of water absorption with time of submersion. Secondly, the rate of water absorption was observed to decrease with increasing compaction moisture content, with the water absorption decreasing from samples compacted at 23% to 28% and then to 33%. This trend was observed from the start of submersion right through to the end of the 10-days submersion period.

When lime was replaced by 50% GGBS, to achieve a blended binder of composition 8%Lime-8%GGBS (1:1 ratio) as shown in Fig. 5.13b, the rate of water absorption was drastically reduced by about half of the water absorption observed in Fig. 5.13a for the lime stabiliser. It was also observed that coupled with the reduction due to lime

replacement, the reduction in water absorption due to increasing specimen compaction moisture content was repeated, similar to the trend observed earlier with the 16%Lime-0%GGBS blended mix composition.

Further replacement of lime with GGBS to 4%Lime-12%GGBS (which is a 1:3 blended ratio) as shown in Fig. 5.13c did not cause much change in the water absorption of the Lime-GGBS stabilized mixture. However, there is still the trend of increasing water absorption with submersion time, as well as a reduction in water absorption with increasing specimen compaction moisture content.

The effect of compaction moisture content is further illustrated in a different format in Fig. 5.14 where at the end of the soaking period of 10 days the samples stabilized with 16%Lime-0%GGBS and at 23% compaction moisture content (Fig. 5.14(a)) absorbed about 17% weight of water as against 15% absorption on increasing the compaction moisture content to 28% (see Fig. 5.14(b)) and finally 10% in Fig. 5.14(c) when the compaction moisture content was further increased to 33%. This trend of reduction in water absorption with increase in the compaction moisture content was observed in all the samples whether there was a replacement of lime or not. However, as shown in Fig. 5.14(a), replacing Lime by 50% GGBS at 23% compaction moisture content resulted in a drastic reduction in water absorption, with a 7% water absorption. Further replacement of lime to 4%Lime-12%GGBS did not result in a significant reduction in further water absorption. On comparing Fig. 5.14(a-c) it may be observed that increasing the compaction moisture content reduced the water absorption in all the cases, with a record low value of 2% water absorption with the 4%Lime-12%GGBS blended mix composition upon stabilization at compaction moisture content of 33%.

Figure 5.15 summarises the comparison of the water absorption of the different blended mixes at the different compaction moisture contents at the end of the 10-days submersion period (after an initial moist curing period of 7 days). The 16%Lime-0%GGBS blended mix showed high water absorption in all the compaction moisture contents, although the

water absorption reduced with increasing compaction moisture content. The samples where lime was replaced at different levels showed reduction in water absorption and a further reduction was also observed when the compaction moisture content was progressively increased.

The results narrated so far are those for the test specimens moist cured for 7-days. This work was repeated with extended curing time of specimens prior to submersion in water. Upon further curing of the samples to 14 days as shown in Fig. 5.16, there was a general reduction in the water absorption with changes in stabiliser blends as observed earlier from the test samples moist cured for 7-days prior to soaking. However, Fig. 5.16(a) which is the result of soaking 16%Lime-0%GGBS still showed high water absorption. The test specimens stabilised at 23% compaction moisture content and 7 days of moist curing (Fig. 5.14(a)) showed high water absorption of 17% as against the same samples soaked after 14 days of moist curing (see Fig. 5.17(a)) that were observed to absorb about 15% of water. The water absorption continued to show reduction with increasing compaction moisture content.

On replacement of the lime content of the stabilizer mix composition with 50% GGBS (8%Lime-8%GGBS), the water absorption was again reduced by over 50%. However, further replacement of lime with GGBS did not contribute much towards the further reduction of water absorption (see Fig. 5.16(b) and (c)). Comparing Fig.5.13 and 5.16 shows the effect of curing age, prior to soaking, on the water absorption profiles of the samples. It was observed that upon increasing the curing age, the water absorption was reduced. At the end of the soaking period, the water absorption reduced with increase in compaction moisture content as well as with increased replacement of lime, as summarised in Fig. 5.17, and more briefly in Fig. 5.18.

Figures 5.19, 5.22, 5.24 and 5.26 show the water absorption for test specimens moist cured for periods longer than 7 days (28, 56, 90 and 150 days of curing). The Figures all show the changes in water absorption during the 10-days soaking period. Figure 5.21

summarises the data in Figures 5.19, 5.22, 5.24 and 5.26, by illustrating the maximum water absorption for each curing period (28, 56, 90 and 150 days of curing), at the end of the 10-day soaking period.

Figures 5.19 and 5.22 suggest that the water absorption magnitudes decreased upon increasing moist curing the test specimens, prior to soaking, from 28 to 56 days. This decrease in water absorption is more clearly illustrated by Figures 5.21 and 5.23, which show the maximum water absorption after the 10-day soaking period. For each test specimen, the water absorption also decreased with increasing specimen compaction moisture content. As for the test specimen composition, the specimens stabilised using lime without any GGBS (16L-0G) showed highest water absorption, while those with highest GGBS content (4L-12G) showed the least water absorption.

Figure 5.24 shows that water absorption increased upon prolonged moist curing (prior to soaking) from 56 (Figure 5.22) to 90 days. On comparing Figures 5.19 and 5.24, the water absorption magnitudes for specimens moist cured for 28 and 90 days are very close. This is quite interesting since the material this time is expected to have attained almost ultimate or residual stabilisation potential. The increase in water absorption at this late stage of curing suggests a changing internal structure (microstructure), while the external volume or structure has already become stable.

Figure 5.26 shows that upon prolonged moist curing from 90 to 150 days prior to soaking, the changing microstructure manifested during the curing stage from 56 to 90 days seen earlier continued to unfold. The change in internal void volume, suggested by increase in water absorption, appears to have reached a turning point at 90 days for specimens' stabilised using lime alone (16L-0G). These specimens show a marginal reduction in water absorption on curing from 90 to 150 days, but still suggest an ultimate open microstructure for lime-stabilised clays. For the test specimens containing GGBS, the water absorption continued to increase beyond 90 days of curing prior to soaking, with specimens with a highest GGBS content (4L-12G) suggesting highest increase and hence most open microstructure. These are the same specimens that showed the highest strength development, suggested a strong open microstructure. It will be interesting to see whether

this microstructure suggested by the water absorption as summarised in the Figures, will be corroborated by the results of water permeability.

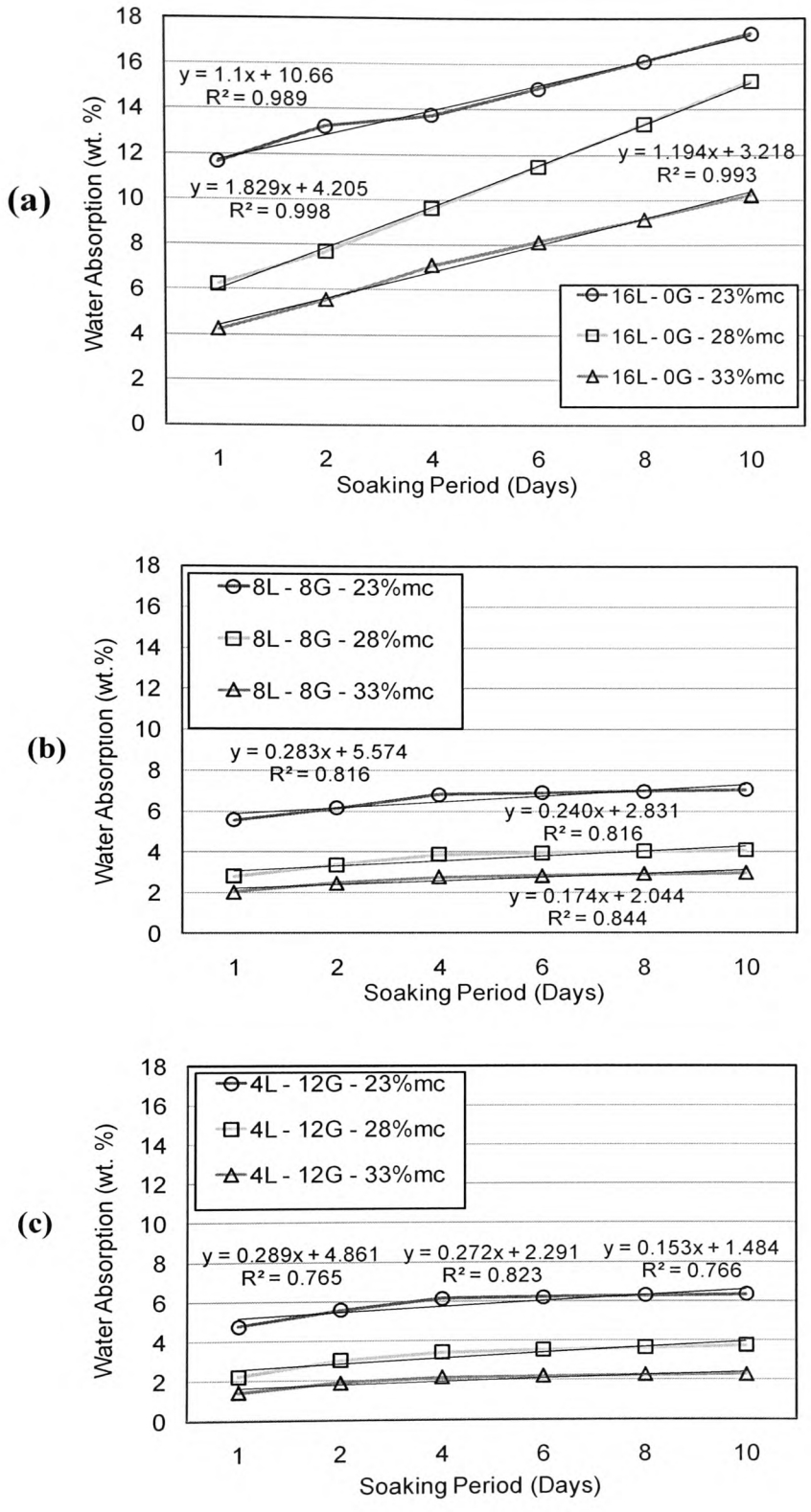


Fig. 5.13 Water absorption profiles of stabilized Lower Oxford Clay after 7-days of moist curing, during soaking in water for 10 days. (a) 16%Lime-0%GGBS (b) 8%Lime-8%GGBS, and (c) 4%Lime-12%GGBS

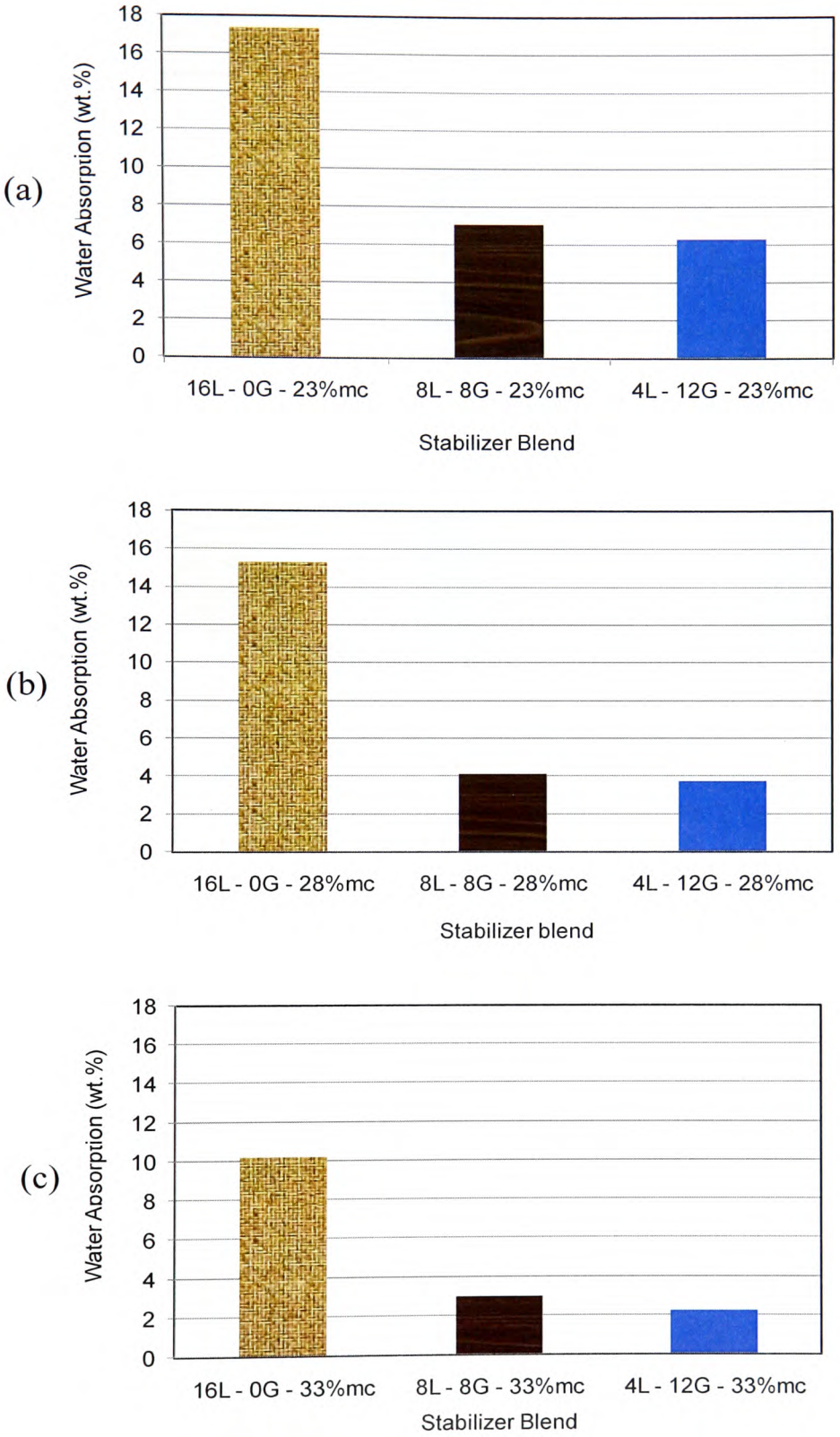


Fig. 5.14 Water absorption profile of stabilized Lower Oxford Clay after 7-days of curing, at the end of the 10-days of soaking in water. (a) at 23% m/c (b) at 28% m/c (c) at 33% m/c

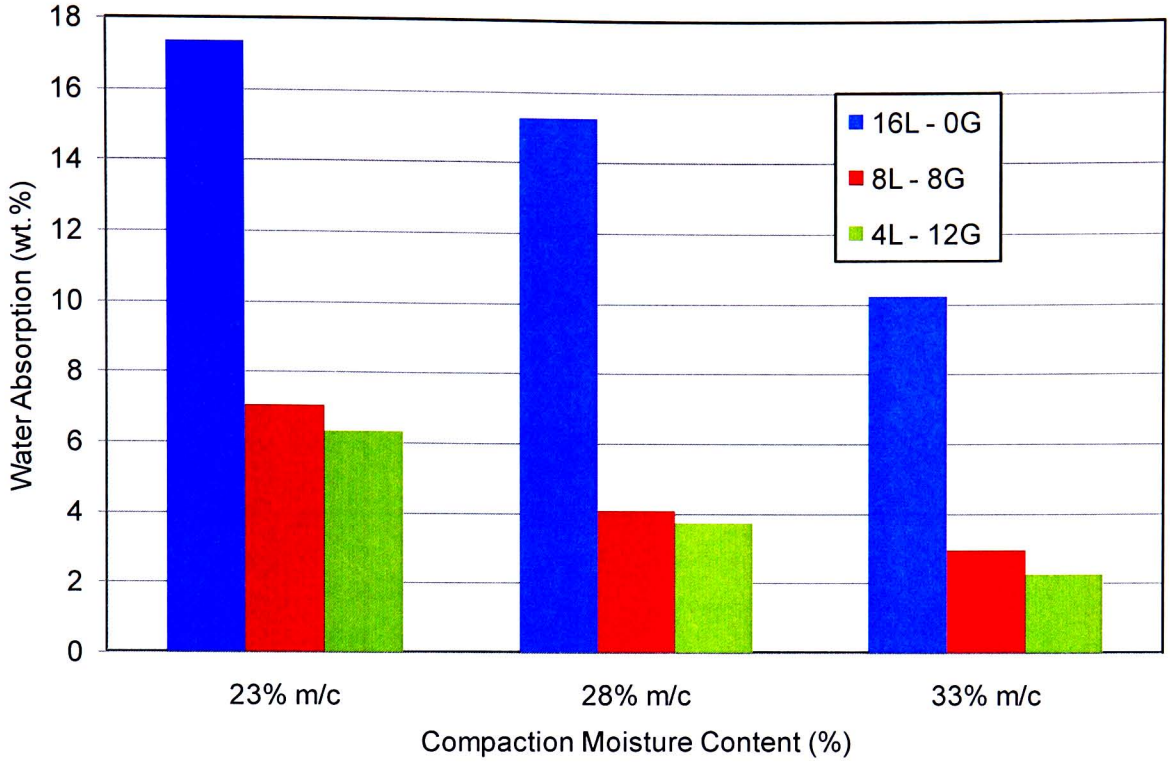


Fig. 5.15 A summary of the Water absorption profiles of stabilized Lower Oxford Clay using different stabilizer blends at different compaction moisture contents, moist cured for 7-days prior to completely soaking in water for 10-days.

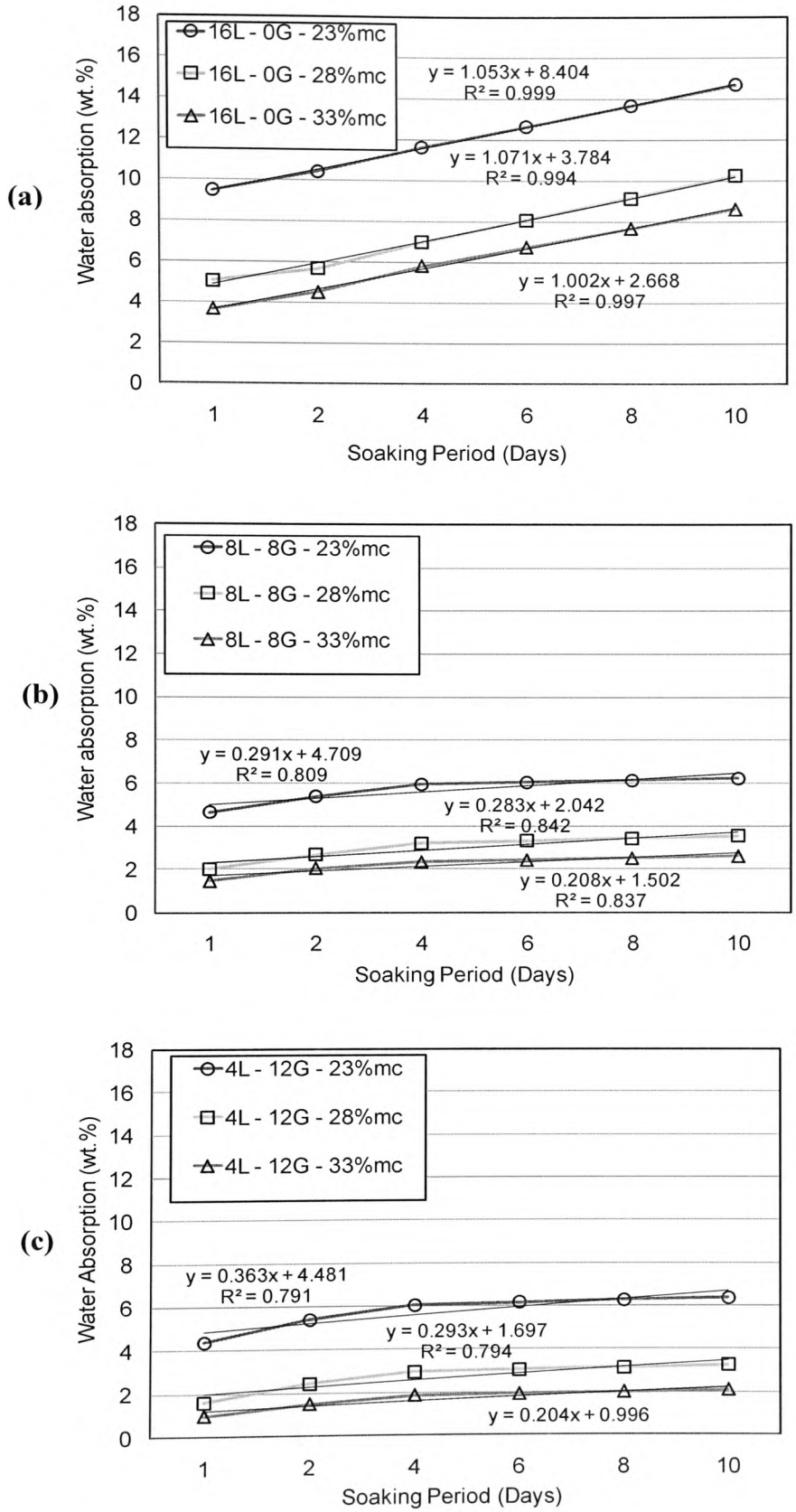
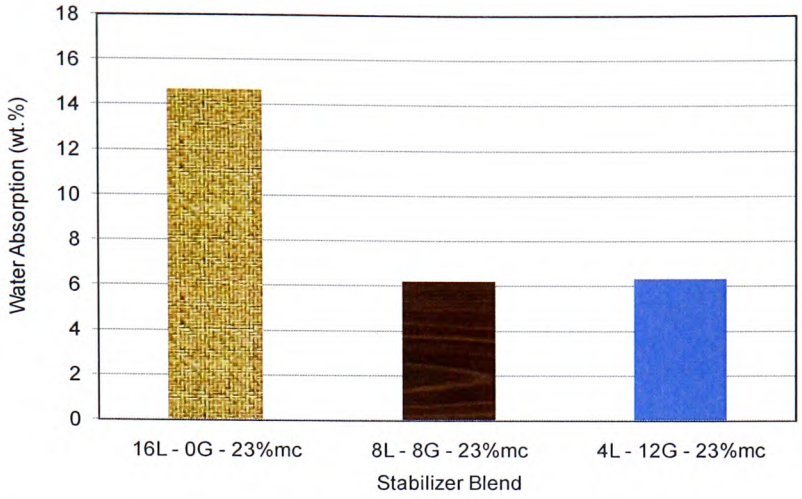
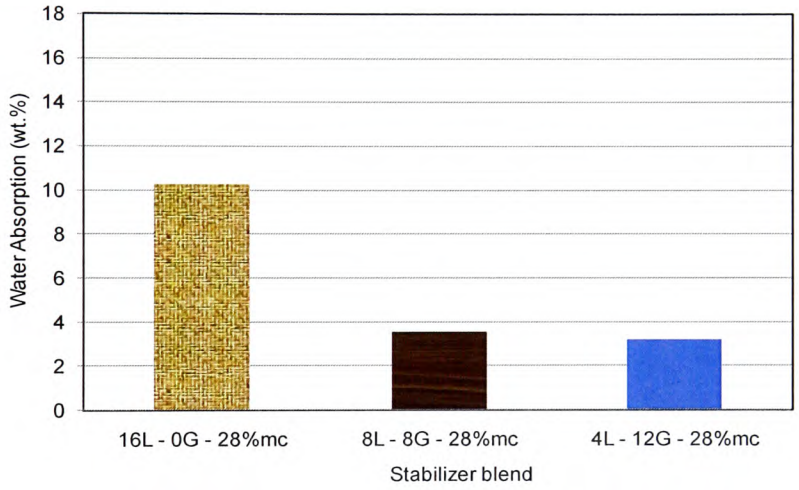


Fig. 5.16 Water absorption profiles of stabilized Lower Oxford Clay after 14-days of moist curing, and 10-days soaking in water. (a) 16%Lime-0%GGBS (b) 8%Lime-8%GGBS (c) 4%Lime-12%GGBS

(a)– 23% m/c



(b) – 28% m/c



(c) -33% m/c

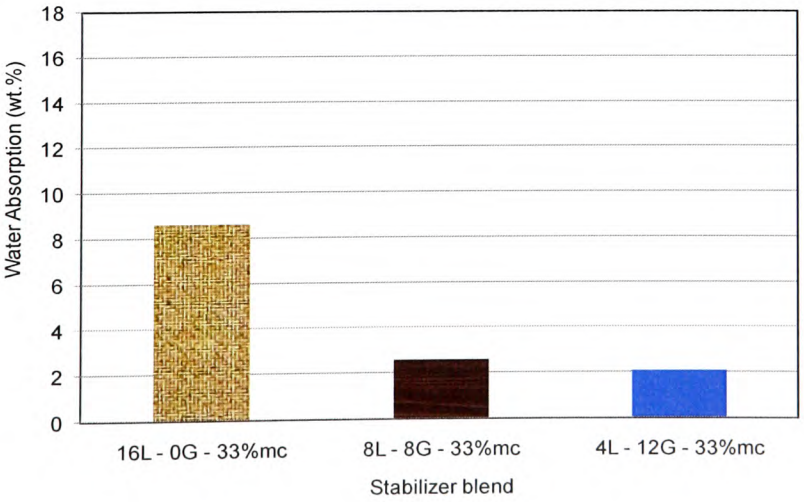


Fig. 5.17 Water absorption profile of stabilized Lower Oxford Clay after 14-days curing and 10-days soaking.

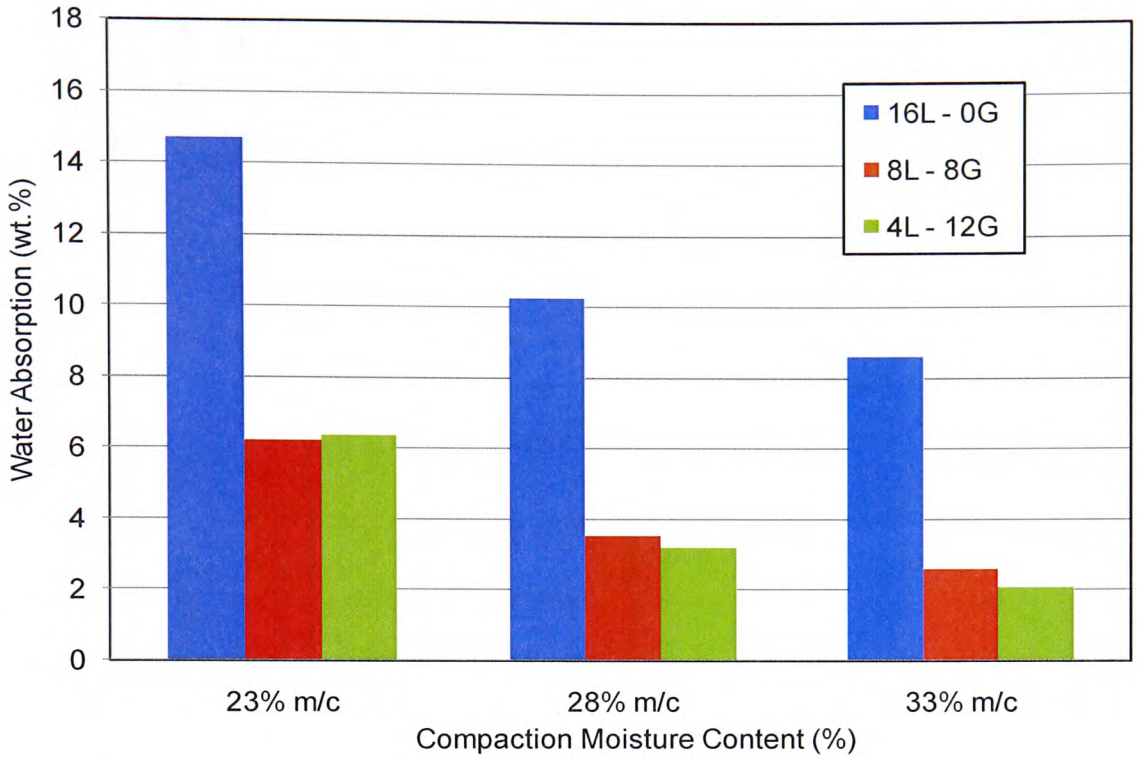


Fig. 5.18 Water absorption profile of stabilized Lower Oxford Clay using different stabilizer blended mix compositions at different compaction moisture content levels, moist cured for 14-days and completely soaked in water for 10-days.

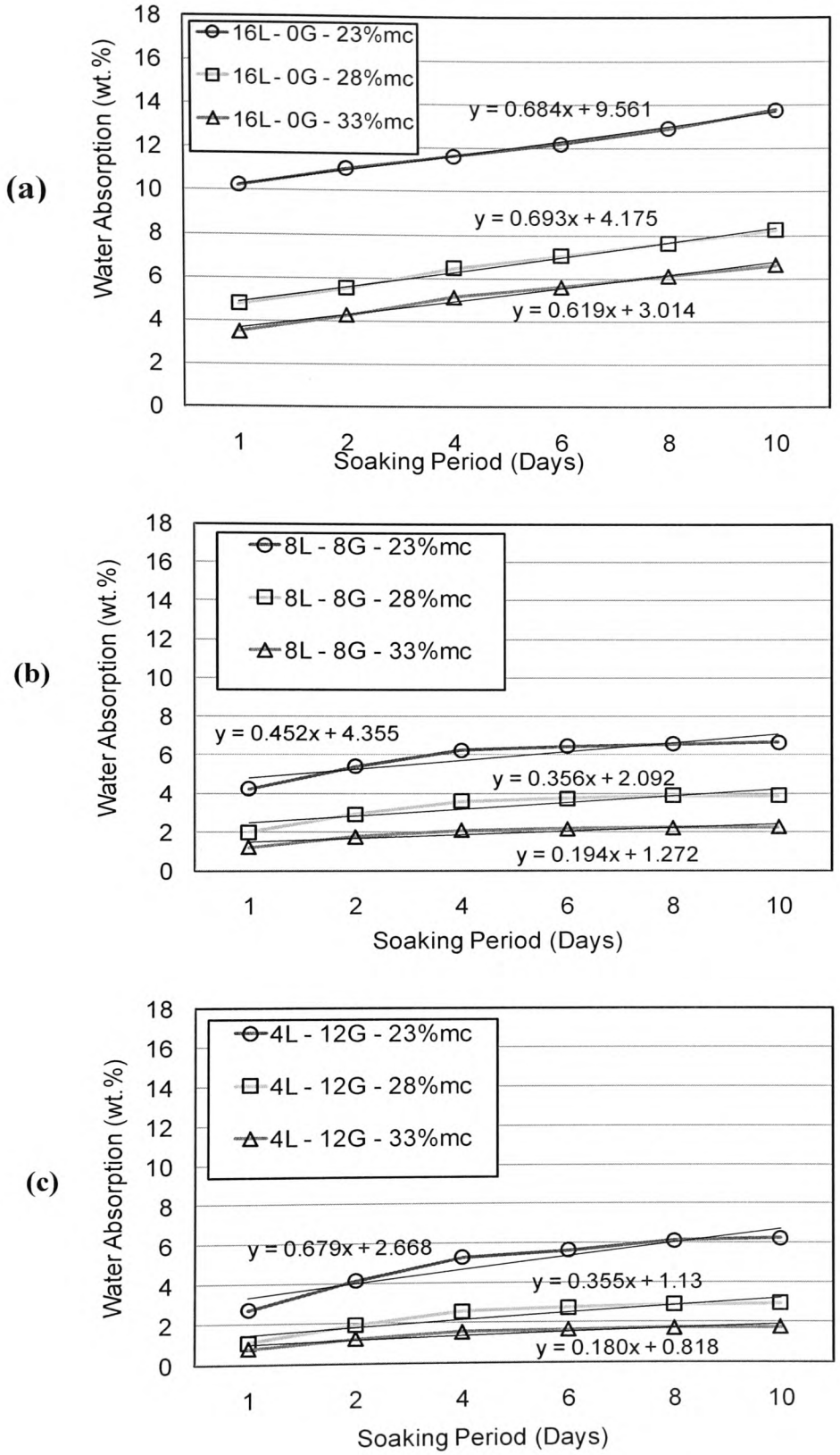
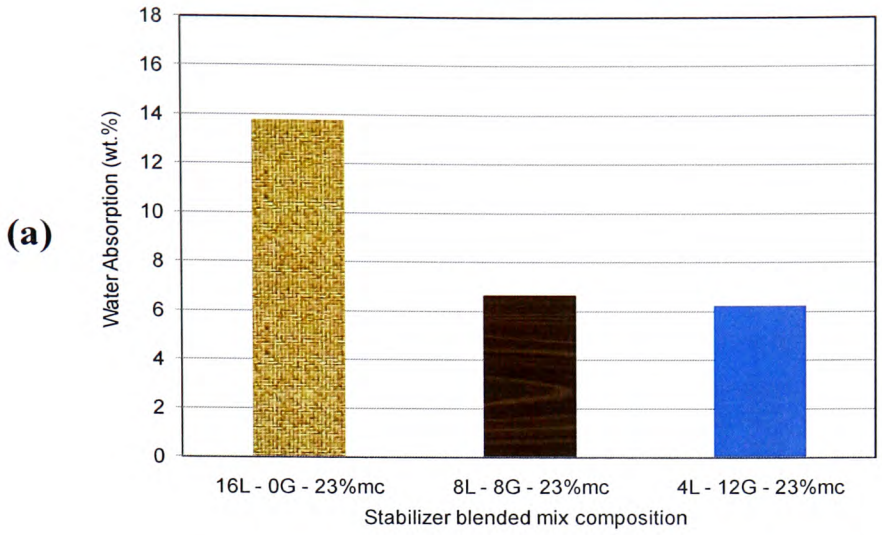
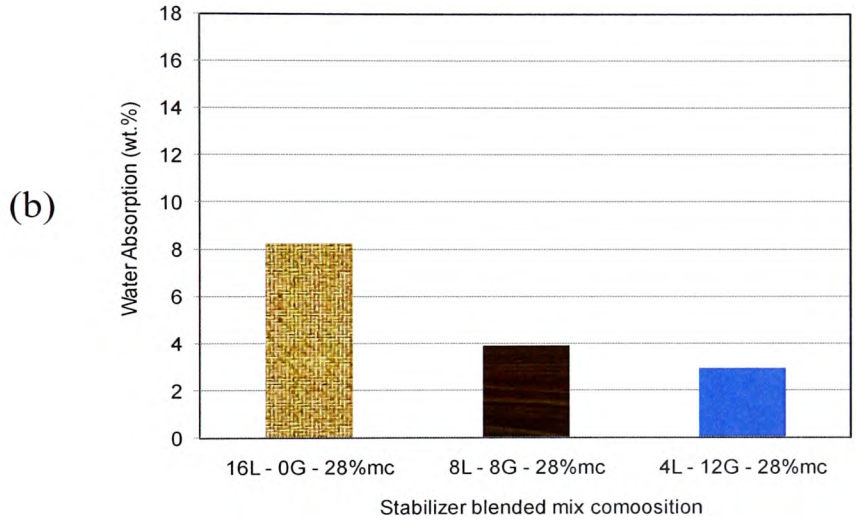


Fig. 5.19 Water absorption profile of stabilized Lower Oxford Clay after 28-days curing and 10-days soaking. (a) 16%Lime-0%GGBS (b) 8%Lime-8%GGBS (c) 4%Lime-12%GGBS





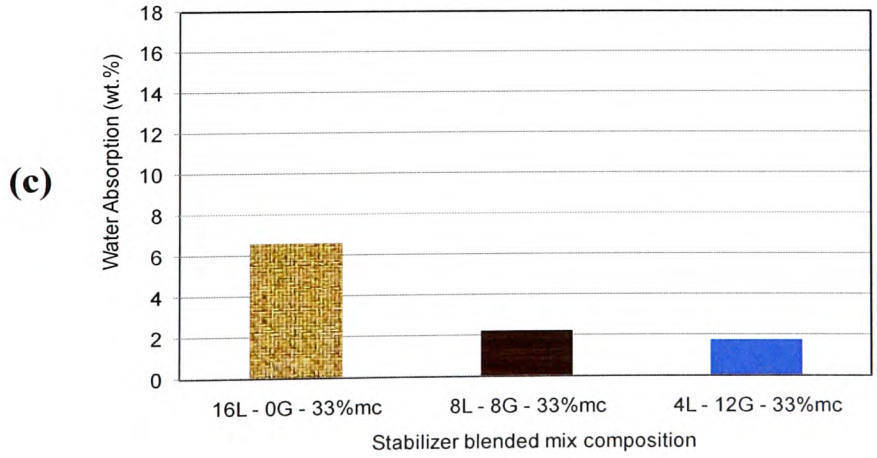


Fig. 5.20 Water absorption profile of stabilized Lower Oxford Clay after 28-days curing and 10-days soaking. (a) at 23% m/c (b) at 28% m/c (c) at 33% m/c

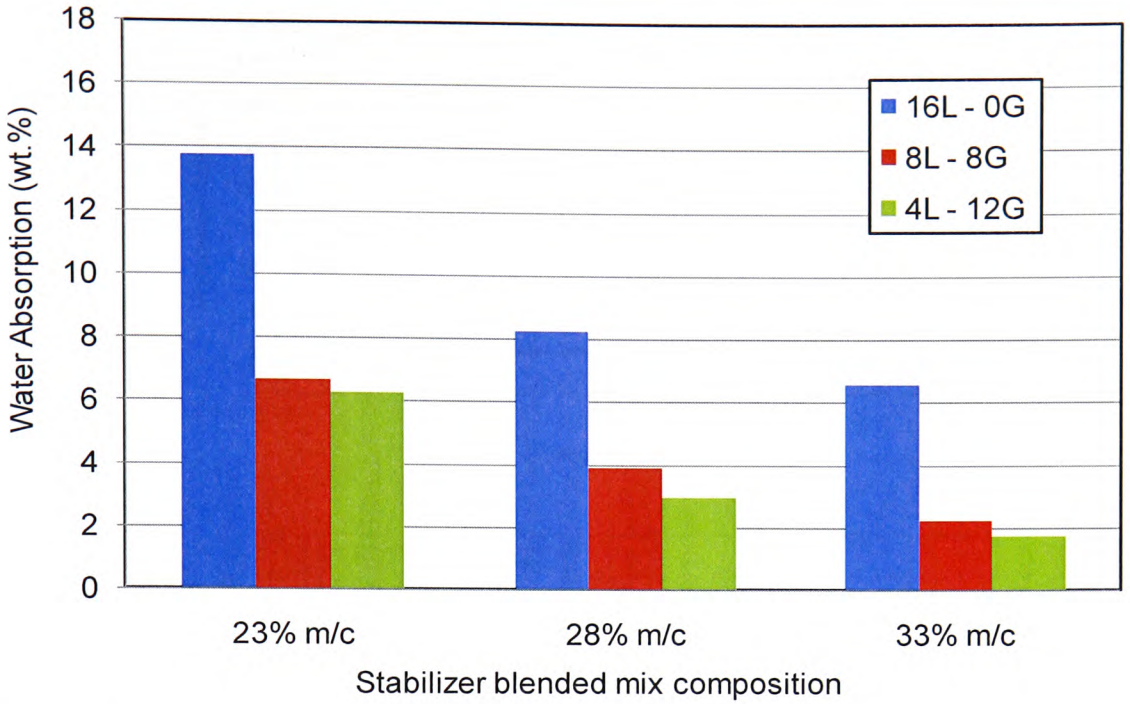


Fig. 5.21 Water absorption profile of stabilized Lower Oxford Clay using different stabilizer blended mix compositions at different compaction moisture content levels, moist cured for 28-days and completely soaked in water for 10-days.

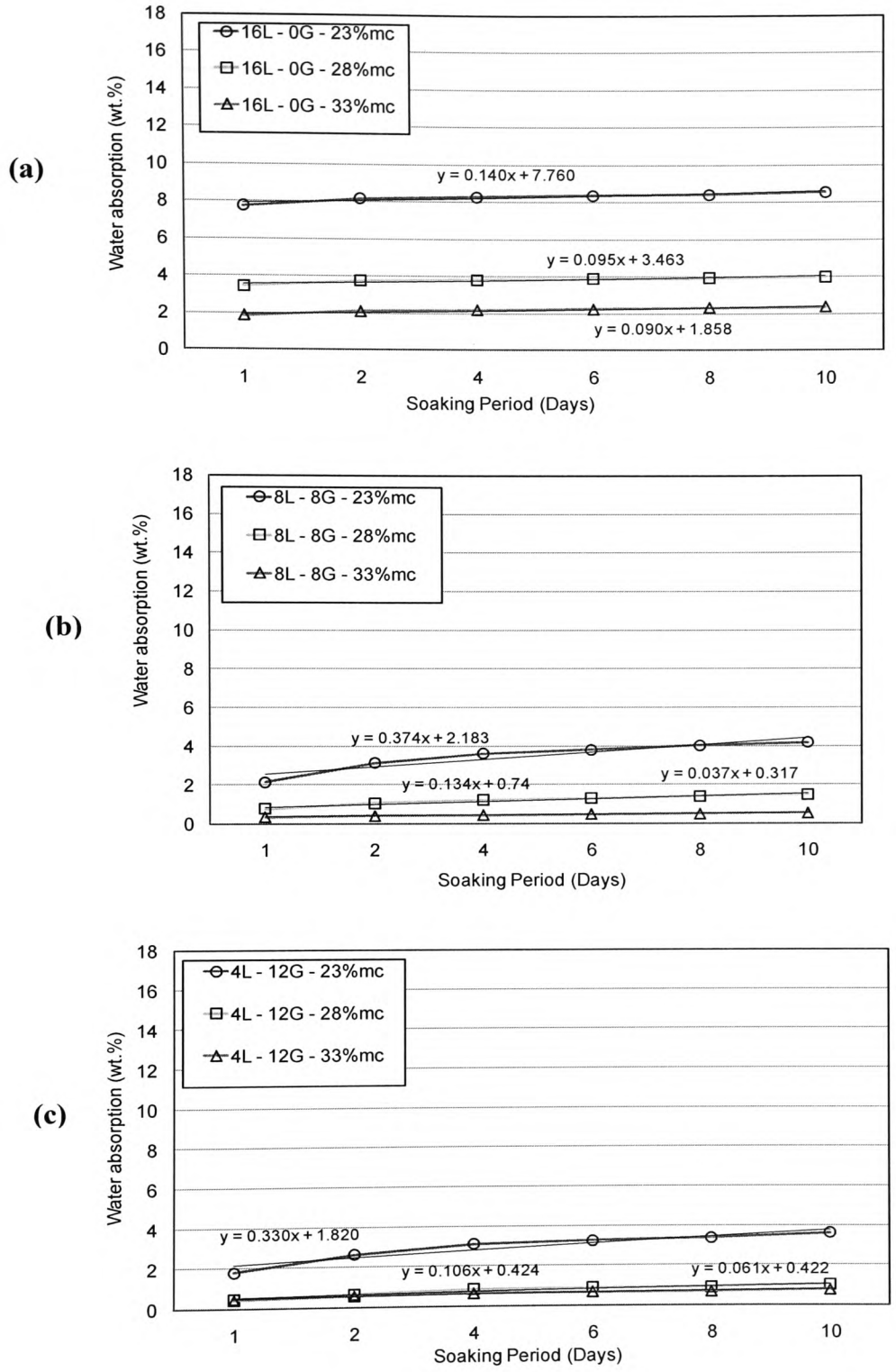


Fig. 5.22 Water absorption profile of stabilized Lower Oxford Clay after 56-days curing and 10-days soaking. (a) 16%Lime-0%GGBS (b) 8%Lime-8%GGBS (c) 4%Lime-12%GGBS

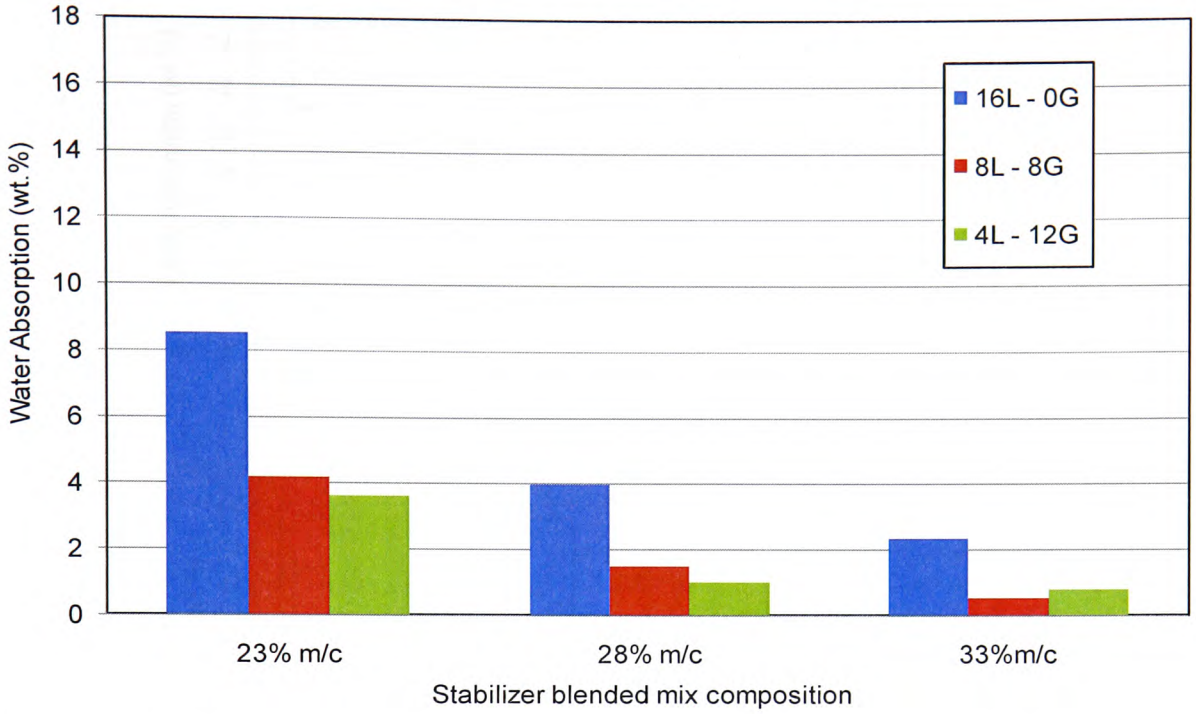


Fig. 5.23 Water absorption profile of stabilized Lower Oxford Clay using different stabilizer blended mix compositions at different compaction moisture content levels, moist cured for 56-days and completely soaked in water for 10-days.

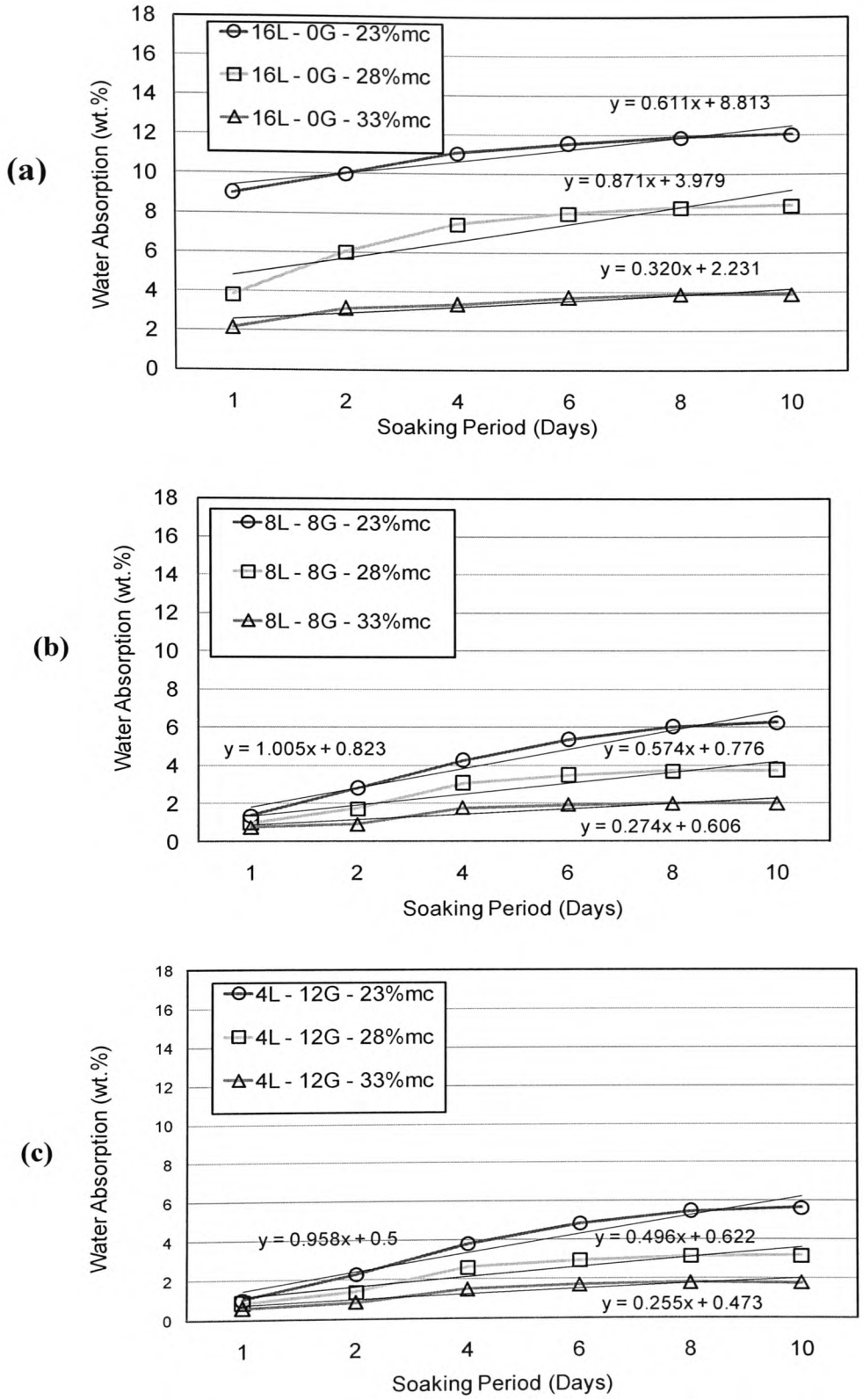


Fig. 5.24 Water absorption profile of stabilized Lower Oxford Clay after 90-days curing and 10-days soaking. (a) 16%Lime-0%GGBS (b) 8%Lime-8%GGBS (c) 4%Lime-12%GGBS

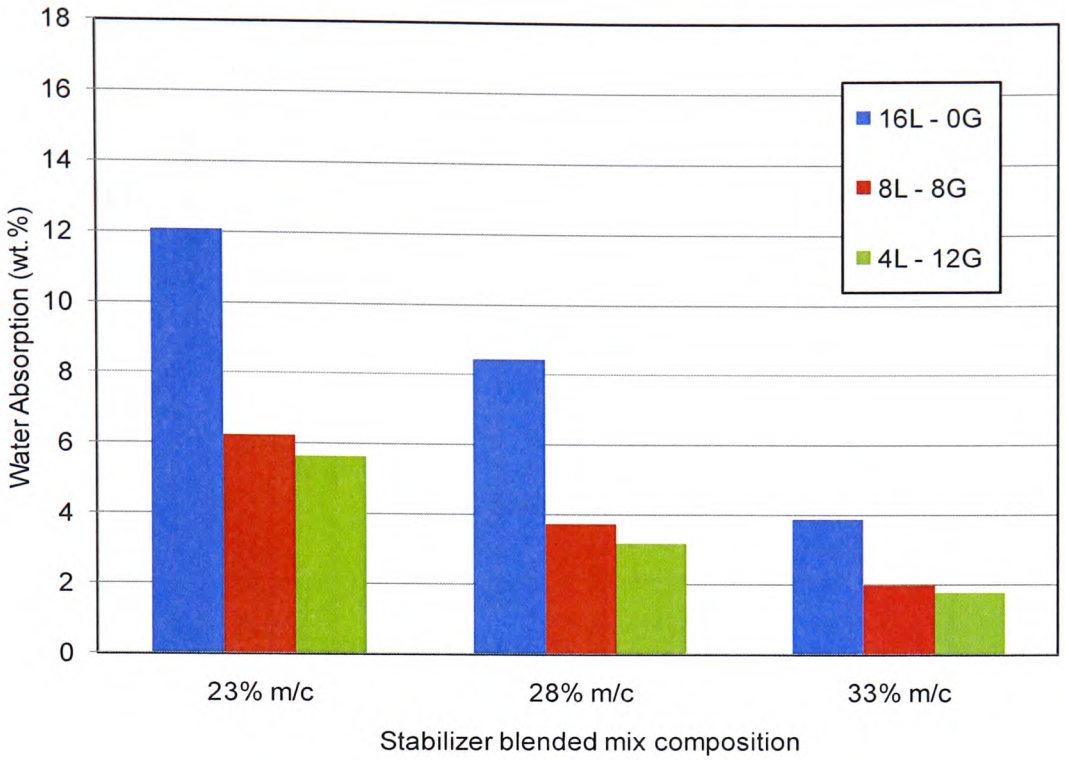


Fig. 5.25 Water absorption profile of stabilized Lower Oxford Clay using different stabilizer blended mix compositions at different compaction moisture content levels, moist cured for 90-days and completely soaked in water for 10-days.

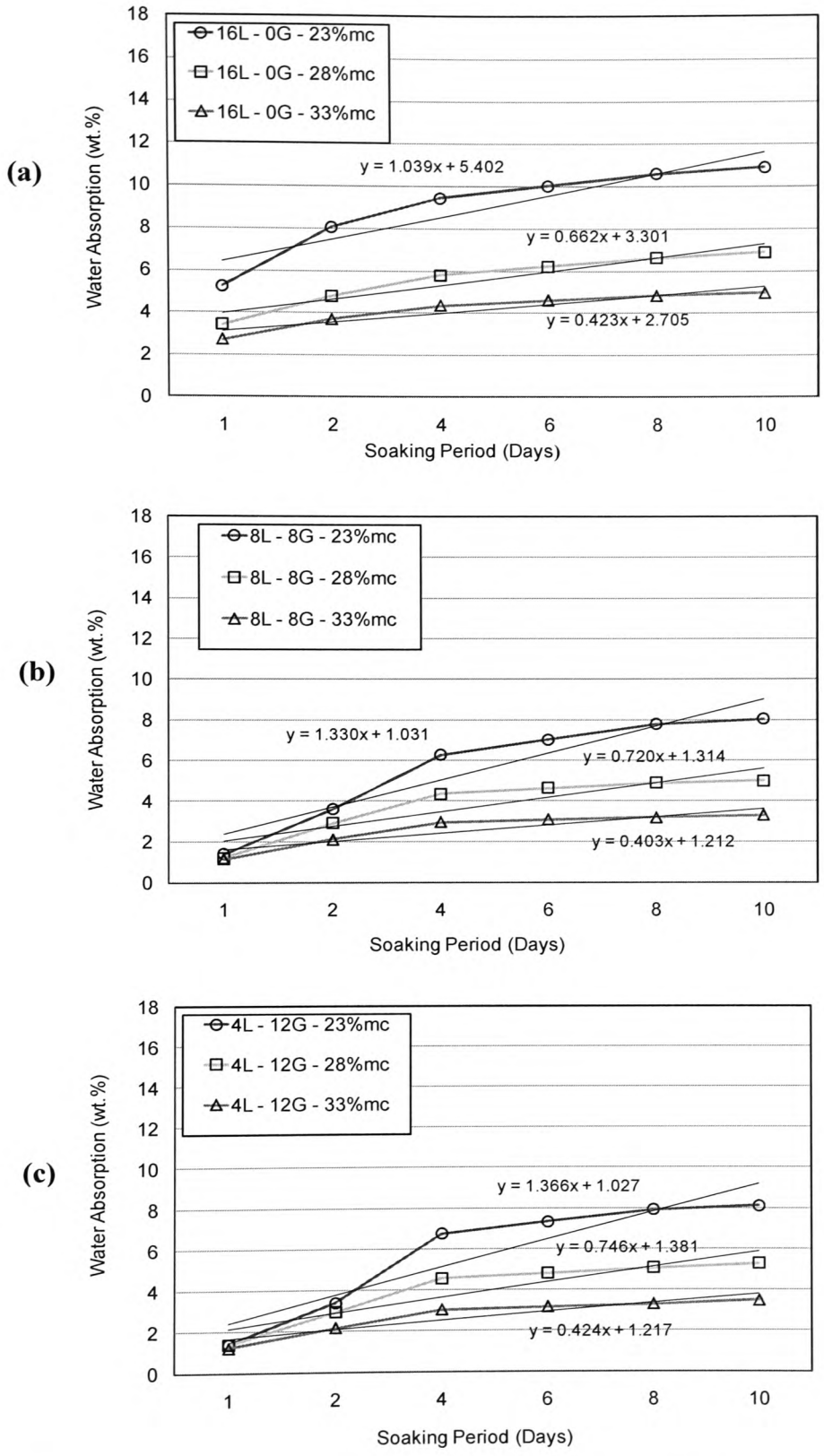


Fig. 5.26 Water absorption profile of stabilized Lower Oxford Clay after 150-days curing and 10-days soaking. (a) 16%Lime-0%GGBS (b) 8%Lime-8%GGBS (c) 4%Lime-12%GGBS

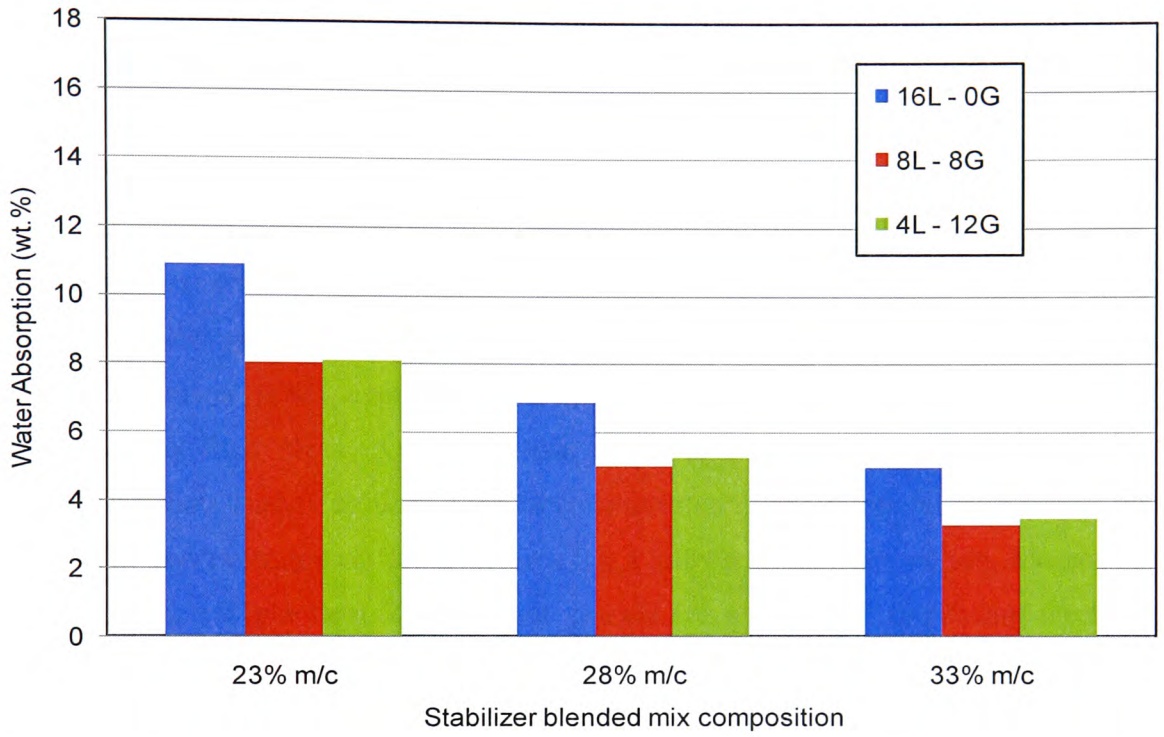


Fig. 5.27 Water absorption profile of stabilized Lower Oxford Clay using different stabilizer blended mix compositions at different compaction moisture content levels, moist cured for 150-days and completely soaked in water for 10-days.

5.4.2 Permeability

Permeability is a measure of the flow of fluids (especially, but not always water) through a porous medium. The hydraulic permeability of subsurface water is an important factor in geotechnical engineering. In road construction, for instance, the effect of water on the performance of the structural layers of a road could be significant and compromise strength and stability. Determination of permeability involves establishing the quantity of water passing through a given cross sectional area, and this is a function of the pressure head acting on the fluid. Traditionally, permeability is measured in Darcy units (but in SI units it is m^3/m^2/day or m/s) where 1 Darcy equals to 10^{-12} m/s. The study of permeability is important as not all pores are used for fluid transmission. There are three main types of void spaces that exist in soils – micro-pores, capillaries and macro-pores. Porous materials are classified into several kinds by their sizes. A microporous material is a material containing pores with diameters less than 2nm. Capillaries are inter-connected micro-pores to form micro vessels, which aid the transportation of fluids under a force known as capillary action. Macropores are cavities which are larger than 50nm. These posses the attribute of increasing the hydraulic conductivity of the soil. The presence of any of these pores affects the intrinsic permeability of a given soil. In determining the permeability of a soil it is important to understand the subjects of absolute and effective porosities; the intrinsic permeability of the material and the degree of saturation.

Absolute porosity: is a measure of the void space in a material. This represents a fraction defining the total volume of voids over the total volume of material. It is possible for a material to have a high porosity but a low permeability, as in clays. This is a function of the effective porosity of the material; it is the fraction of the absolute porosity in which fluid flow is effectively taking place.

Intrinsic and absolute permeabilities: these are terms highlighting the fact that the permeability in perspective is that of the material structure and not of the fluid.

Figures 5.28-5.32 show the permeability data for stabilized Lower Oxford Clay after moist curing periods of 7, 56 and 90 days. The determination of the quantity of water passing through a granular soil material at a given time period is traditionally assessed in the laboratory by either the falling head or the constant head permeability methods discussed in

Section 4.6 of Chapter 4. However, in the current research, it was difficult to carry out permeability measurements of stabilised soil as there is no standard equipment for this. The results presented in the current research work are the author's best attempt at determining the flow of fluid (water) through a stabilized soil material. A Perspex Confined Permeability Testing apparatus with a 25mm aperture was used to tailor to the needs and limitations of the current research (see Section 4.6.1 of Chapter 4). Cylinder specimens of dimensions 50mm in diameter and 100mm long were used. The hydraulic conductivity of water through a stabilized soil material medium is different from that through natural granular soil. In the case of a granular soil, evaporation is the most prominent factor to contend with in ensuring an accurate permeability measurement, whereas in stabilized soil materials, due to the different reactions of the stabilizing materials, there are different demands for water. Water is needed for the pozzolanic reactions involving clay minerals, the stabilizer used. In the current research work, lime and GGBS were used as stabilisers. In carrying out permeability experiment on a stabilised soil, some of the water passing through the system is used in enhancing strength development reactions, thus further densifying the system and resisting the subsequent flow of the fluid. On the formation of C-S-H gel the matrix structure in a stabilized system is enhanced and this gel also possesses the ability to block micro pores thereby reducing the intrinsic permeability of the material formulations.

Figure 5.28 shows the permeability of stabilized Lower Oxford Clay using 16%Lime-0%GGBS blended stabilizer at different moisture contents and varying ages of curing. From Fig. 5.28a which is the permeability profile of stabilized LOC cured for 7 days, it is clear that without any replacement of lime about 180 cm^3 of water passed through the 16%Lime-0%GGBS stabilized LOC compacted at 23% compaction moisture content and about 100 cm^3 passed through LOC stabilized with the same blended mixture at 28% moisture content after 148 hours of observation period respectively. On further increment of the compaction moisture content from 28% to 33%, a significant reduction in specimen permeability was achieved.

When the period of moist curing specimen was increased to 56 days as shown in Fig. 5.28(b), the samples compacted at 23% compaction moisture content still showed the highest

permeability results, with values above 80 cm^3 at the end of the 148-hours of observation time. This was followed closely by the samples compacted at 28% moisture content and then those at 33% compaction moisture content which were about 20 cm^3. This is similar to the trend observed in Fig. 5.28(a) above where the permeability reduced with increasing compaction moisture content. It is observed that the rate of flow of water through the stabilized medium was reduced significantly after the extended moist curing period.

Further increase in the moist curing time to a 90 days period, affected the samples compacted at low compaction moisture contents significantly. This tends to repeat what went before after 7-days moist curing. However, at increased compaction moisture contents of 28% and 33% the permeability was observed to be minimal, the established trend still remained that of reducing permeability with increasing compaction moisture content.

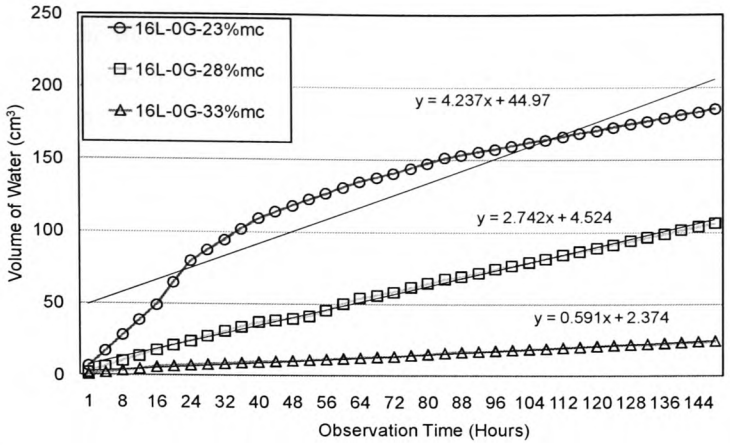
Upon the replacement of lime with GGBS at a ratio of 1:1 (8%Lime-8%GGBS) as shown in Fig. 5.29, there was a significant reduction in the rate of permeability as shown by the results of the different samples at the various compaction moisture contents. This resulted in reducing the scale from a maximum of 200 cm^3 as was in Fig. 5.28 to a maximum of 120 cm^3 in Fig. 5.29. However, the trends observed are still congruent with those observed earlier where permeability reduced with increasing compaction moisture content as well as with increasing moist curing ages. From Fig. 5.29(a) the samples compacted at 23% compaction moisture content recorded a high permeability after 7 days of moist curing. The permeability was very low with the 28 and 33% compaction moisture contents. Upon extended moist curing to 56 days (Fig. 5.29(b)) the permeability result was reduced to about half of what was achieved after 7 days. Further reduction was still realised when these samples were moist cured further for a 90 days period (see Fig. 5.29(c)).

Further replacement of lime at a ratio of 1:3 (4%Lime-12%GGBS) as shown in Fig. 5.30 also showed a general reduction in permeability. This is evident from the reduction in scale from 120 cm^3 maximum in Fig. 5.29 to a maximum scale of 80 cm^3 in Fig. 5.30. However, for the individual specimens, those compacted at 23% compaction moisture content still showed

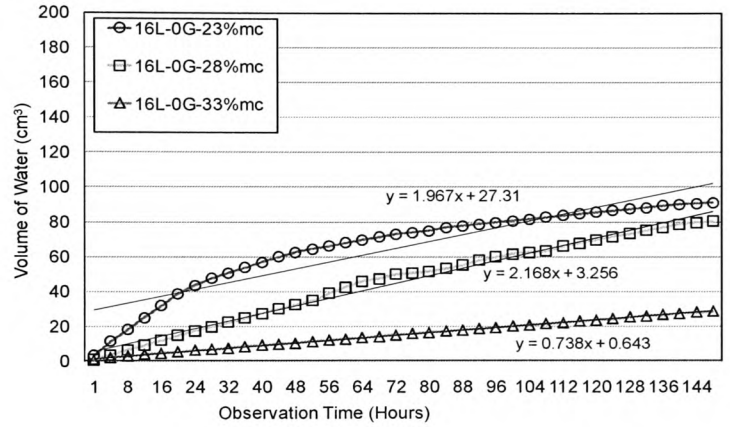
highest permeability records as against the 28% and 33% where the permeabilities were almost negligible especially after an extended moist curing period of 90 days. From Fig. 5.30(a) the samples compacted at 23% compaction moisture content showed about 65 cm^3 of fluid flowing through them after the observation period of 148 hours. This same mix composition after a moist curing period of 56 days reduced the rate of fluid flow drastically by about 50%, however, extending the curing period to 90 days did not have any significant effect on these samples compared to the 56 days permeability result.

Fig. 5.31 shows the permeabilities of LOC stabilized using lime and lime-GGBS blended binders after moist curing for 7, 56 and 90 days after an observation period of 148 hours. The trend of reducing permeability with increasing compaction moisture content is well visualized (see Fig. 5.31(a)). The other trends that are also prominently observable are the effects of moist curing age which reduces the permeability, as well as the effect of lime replacement. These are further summarised in Fig. 5.32 which compared the effect of stabilizer composition, age of moist curing and the variation in compaction moisture content.

(a) – 7 days



(b) – 56 days



(c) – 90 days

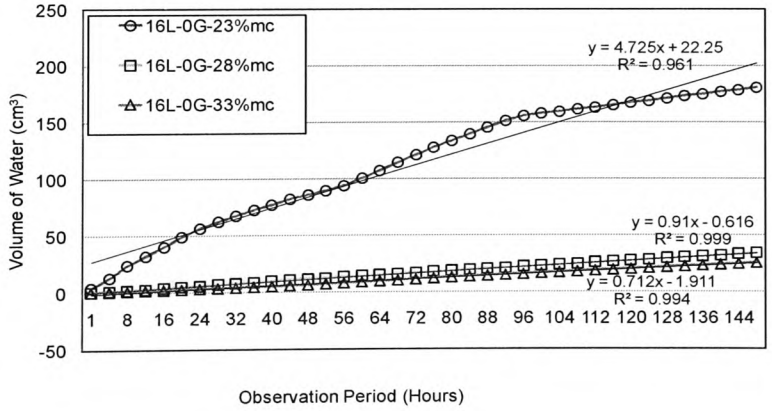
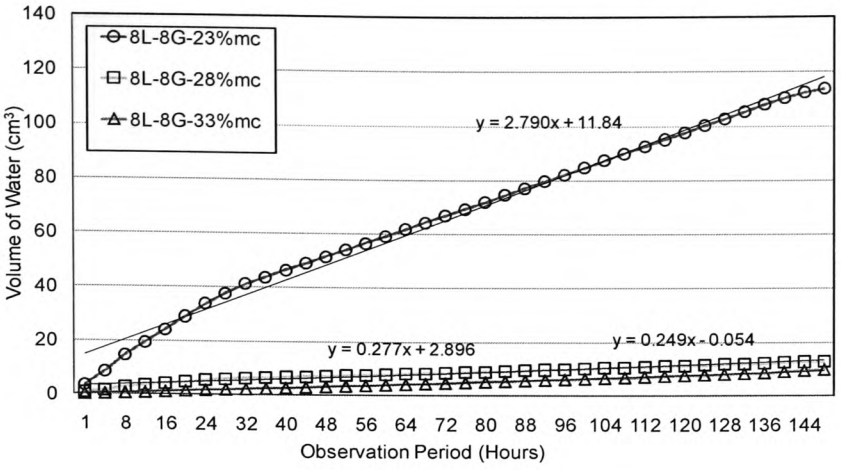
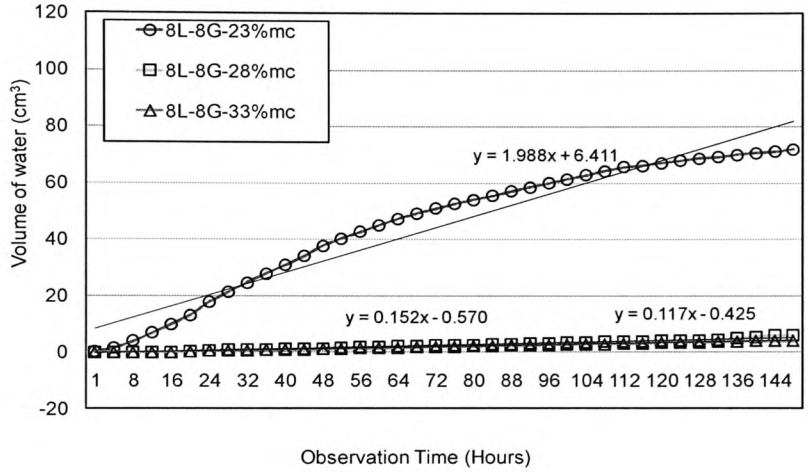


Fig. 5.28 Permeability data for Lime-stabilised Lower Oxford Clay after different curing periods, for test specimens prepared at different compaction moisture contents.

(a)



(b)



(c)

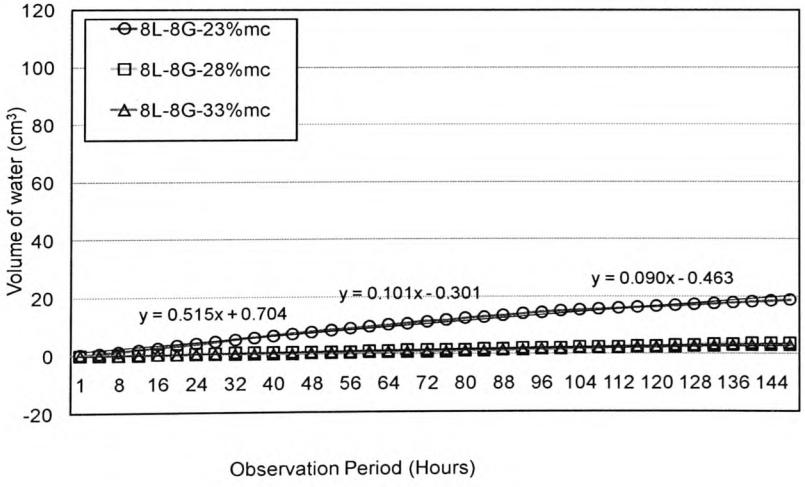


Fig. 5.29 Permeability data for Lower Oxford Clay stabilised using a 1:1 Lime-GGBS blend and at different compaction moisture contents, after moist-curing for different periods. (a) 7-days (b) 56-days, and (c) 90-days.

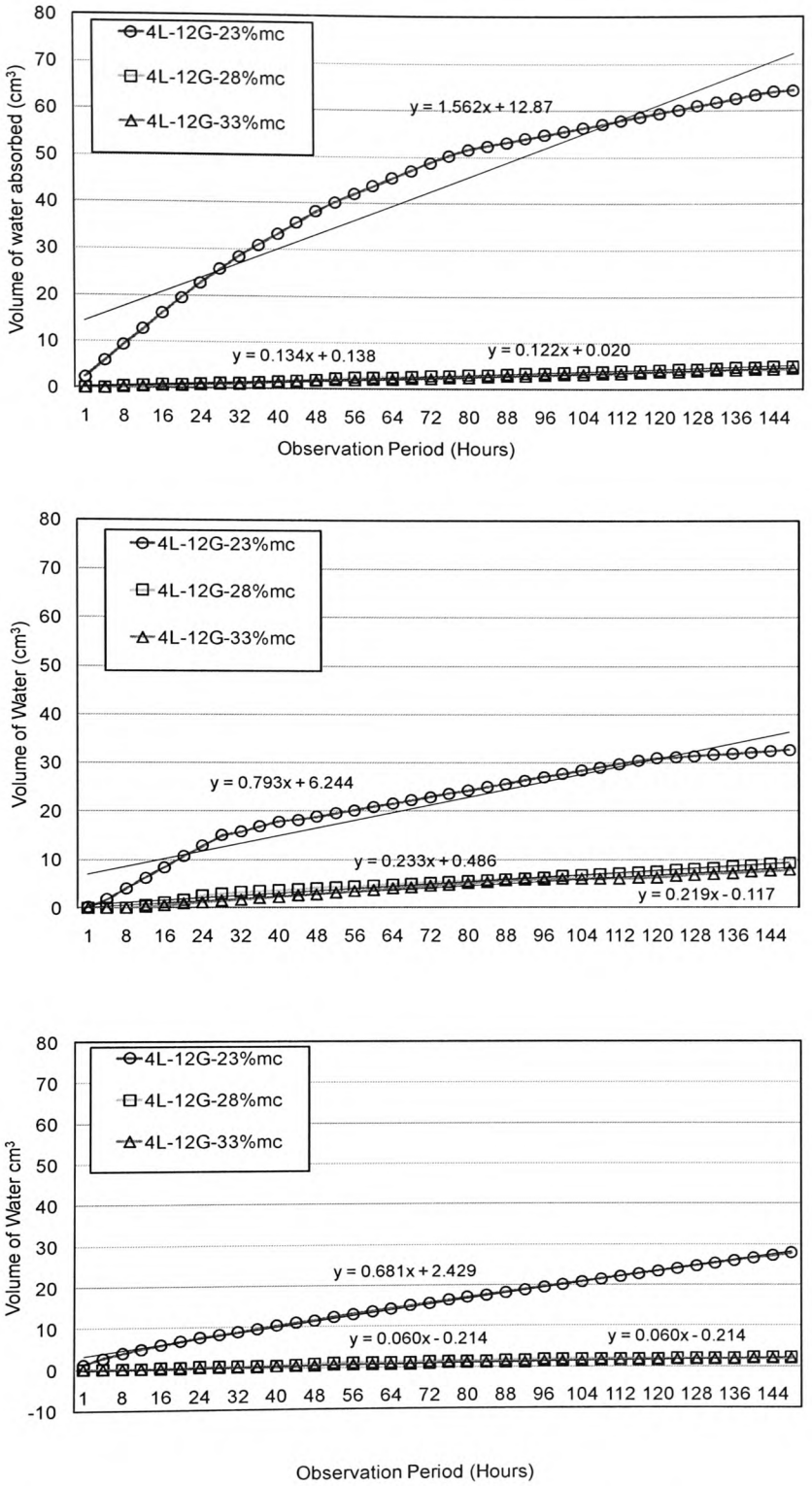
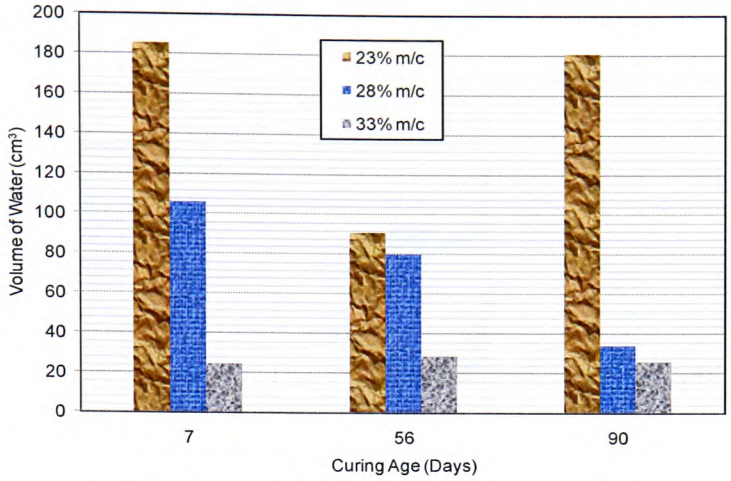
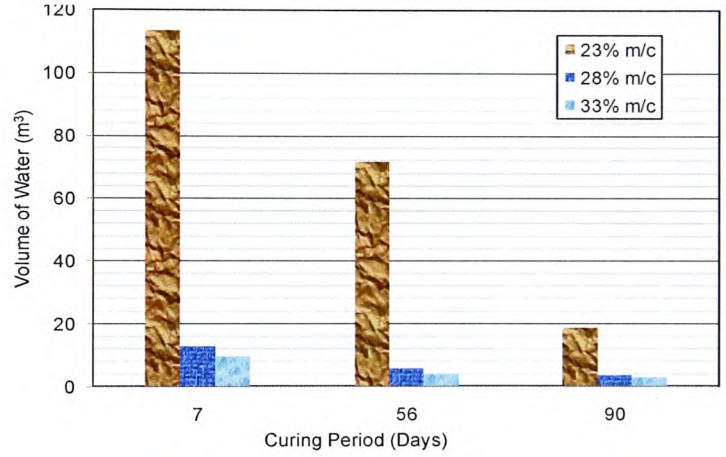


Fig. 5.30 Permeability data for Lower Oxford Clay stabilised using a 1:3 Lime-GGBS blend and at different compaction moisture contents, after moist-curing for different periods. (a) 7-days (b) 56-days, and (c) 90-days.

(a) - 16%Lime-0%GGBS



(b) – 8%Lime-8%GGBS



(c) – 4%Lime-12%GGBS

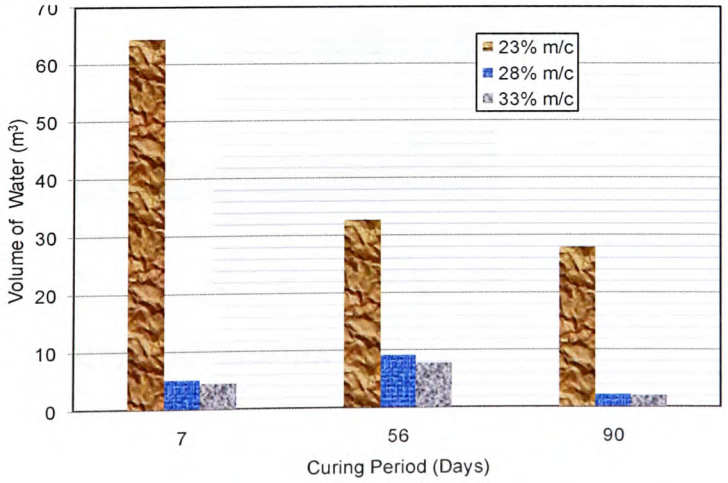


Fig. 5.31 Permeabilities of LOC stabilised using lime and lime-GGBS blended binders after moist curing for 7, 56 and 90 days after an observation period of 144 hours.

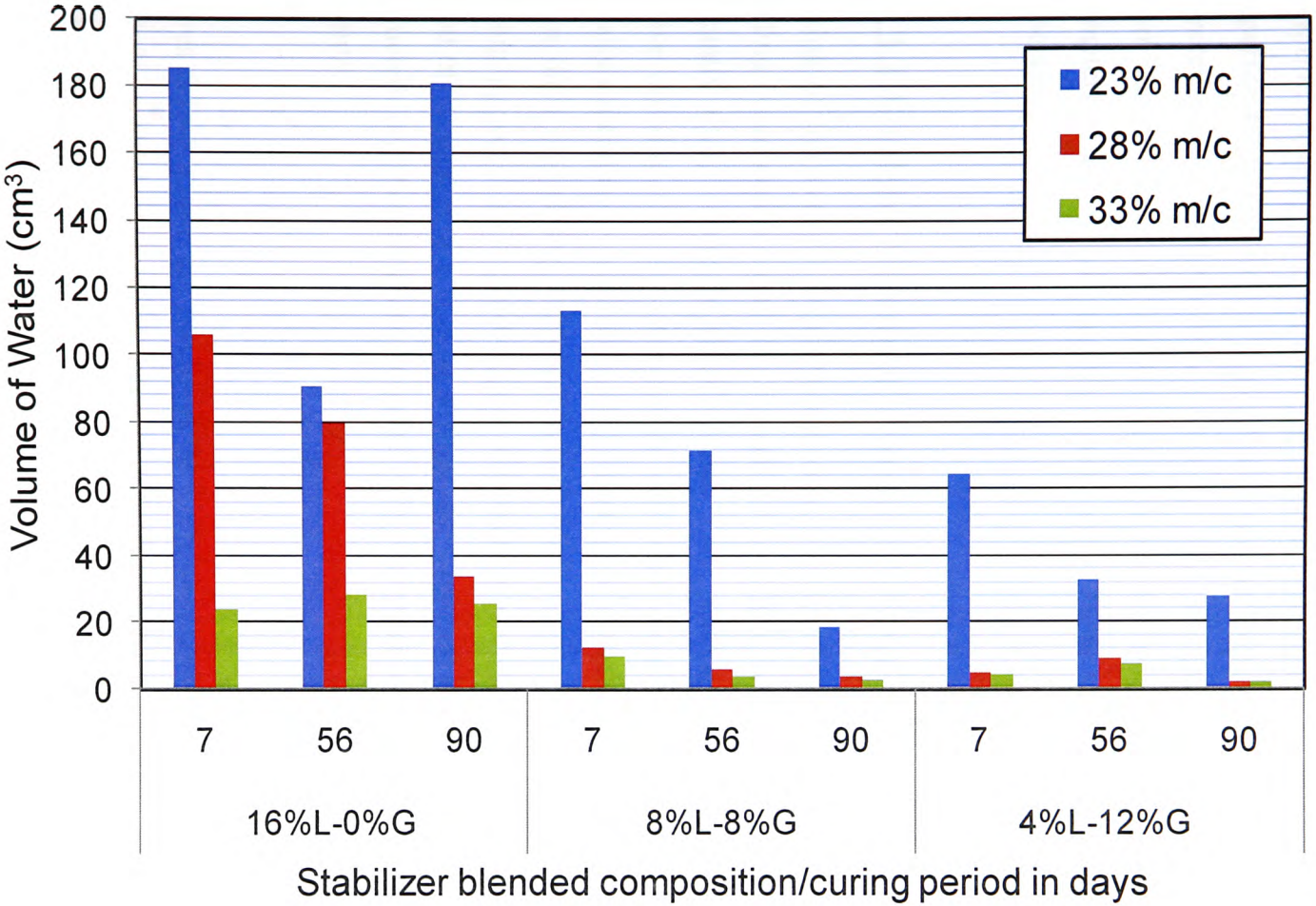


Fig. 5.32 Summary and comparison of the different permeabilities of the different blended mixes after moist curing for 7-, 56- and 90-days.

5.4.3 Effect of Water Absorption and Permeability on

The process by which water passes through a material is characterized as the material's permeability whereas the ability of the material to hold up water in its pores is due to its absorption property. In determining the hydric properties of a material the water absorption and permeability criteria are important factors as they regulate the ingress and percolation characteristics of water within a system. And the amount of water within a stabilized system can influence the Soaked Strength, the Volume Stability and the Compressibility parameters of a stabilized system significantly. These are the subjects for consideration in this section.

5.4.3.1 Soaked Strength (Soaked UCS)

Soaking of the various samples was anticipated to be a key durability testing procedure as flooding could compromise the strength of soil. It is even more critical for low strength soils especially those found in flood plains. Two soaking regimes were established; the first involve soaking two set of samples with one completely soaked in water and the other partially soaked up to 10mm above the bottom of samples. The samples were kept in water for a four days period and UCS test was carried out on the samples at the end of the 4-days soaking period. The second regime involved submerging samples in water for an extended period of 10-days and assessing strength development consequently. These assessments were conducted on the different blended mix compositions at varying compaction moisture contents.

Figures 5.33-5.35 are the results of the UCS tests conducted on the various samples soaked for 4-days either completely designated as (CSS) or partially (PSS) compared with the unsoaked or control (OS). The general trend observed is that of increasing strength with curing age even under soaked environment. Figure 5.33 shows the UCS results of stabilizing LOC with 16%Lime-0%GGBS at varying compaction moisture contents and moist curing ages of 7, 14, 28, 56 and 90days during partial and complete immersion in water. Upon stabilization of LOC with 16%Lime-0%GGBS at 23% compaction moisture content as shown in Fig. 5.33a low strength development was observed. The control sample which was unsoaked started with a 7-day compressive strength of about 1400 kN/m^2 and increased

gradually to a peak of about 1800 kN/m^2 after 56 days. There was then a little drop in strength between the 56 and 90 days strength evaluation periods.

Soaking the samples in water partially (PSS) resulted in a significant loss in compressive strength although there was still the trend of strength development with curing age even under soaked condition. The strength rose from a 7 days strength value of about 250 kN/m^2 to a 90 days compressive strength of about 700 kN/m^2. Comparing the strength of the unsoaked and soaked samples of the 16%Lime-0%GGBS at 23% compaction moisture content after 7 days of moist curing, over 1000 kN/m^2 compressive strength was lost and after 90 days about 900 kN/m^2. When the samples were completely submerge in water for 4-days there was no significant loss in strength from that achieved for partial immersion. However, the strength development was still increasing with the age of moist curing.

Increasing the compaction moisture content (see Fig. 5.33b) for the unsoaked samples it was observed that there was a decrease in the start up strength at 7 days relative to that of 23% compaction moisture content at the same age. However, after 90 days moist curing higher strength was observed with the 28% m/c which was above 2000 kN/m^2 against 23% m/c which was about 1600 kN/m^2 (Fig. 5.33). At this higher compaction moisture content of 28%, the loss in strength upon soaking at the early ages of strength development is reduced, with about 700 kN/m^2 lost in 7 days. The trend of increasing strength with curing age still persists. Further increase in compaction moisture content to 33% resulted in low strength development and reduced loss in strength upon soaking, but all the trends observed from the samples at 23% and 28% were repeated.

When lime was replaced by 50% GGBS (8%Lime-8%GGBS) as shown in Fig. 5.34 there was a significant positive influence on the strength development. All the samples showed a trend of increasing strength with moist curing age. There is another trend of decreasing strength with increasing compaction moisture content. Finally, the strength was observed to decrease with increased severity of environmental impact (partial and complete submersion in water) and the loss of strength upon submersion was very minimal demonstrating the influence of

GGBS. Increased replacement of lime to 4%Lime-12%GGBS as shown in Fig. 5.35 still offered an increase in the overall strength development and the reduced loss in strength suffer by lime systems upon submersion in water.

Fig. 5.36 is the Unconfined Compressive Strength of stabilised LOC using different blended binders after the ultimate moist curing age of 90-days and 4-days of partial and complete submersion in water, compared with unsoaked strength of equivalent curing ages. Without any replacement of lime the strength increased from 23% m/c to peak at 28% m/c then reduced at 33% m/c. The highest strength observed was about 2000 kN/m^2 when the mixture was stabilised at 28% compaction moisture content (see Fig. 5.36a). The loss in strength was observed to increase as the severity of environmental impact increased. At 23% compaction moisture content 16%Lime-0%GGBS (OS) samples dropped from a strength value of about 1600 kN/m^2 to about 700 kN/m^2 upon partial soaking and then to 500 kN/m^2 upon complete submersion in water. Similarly, at 28% compaction moisture content the 16%Lime-0%GGBS (OS) samples at strength of above 2000 kN/m^2 reduced to about 750 kN/m^2 upon partial soaking and a further reduction of about 250 kN/m^2 when they were completely soaked. This trend also was observed for the 33% compaction moisture content. Replacing the lime component by 50% GGBS as shown in Fig. 5.36b brought about a drastic improvement in strength development. In this case the overall trend is one of reduction in strength with increasing compaction moisture content. Contrary to the 16%Lime-0%GGBS the 8%Lime-8%GGBS showed low strength loss upon soaking. The unsoaked UCS (OS) samples of the 8%Lime-8%GGBS blended mix at 23% compaction moisture content showed a strength of about 4300 kN/m^2 which upon partial soaking reduced to 3500 kN/m^2 and on complete soaking reduced to about 2750 kN/m^2. At increased compaction moisture content the strength of the OS samples reduced to about 3700 kN/m^2 and to 3000 kN/m^2 upon partial soaking then to about 2700 kN/m^2 on complete immersion. Whereas the 33% m/c recorded about 2700 kN/m^2 for the OS, about 2400 kN/m^2 for the PSS and then about 2100 kN/m^2 for the CSS (see Fig. 5.36b).

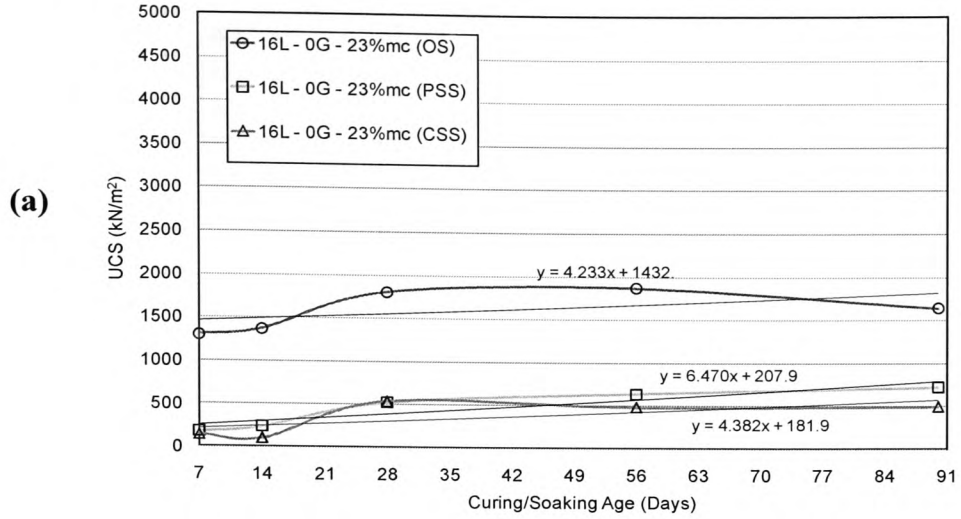
On further replacement of lime to 4%Lime-12%GGBS as illustrated in Fig. 5.36c the strength development was further enhanced. It is also observed that strength loss was higher at low

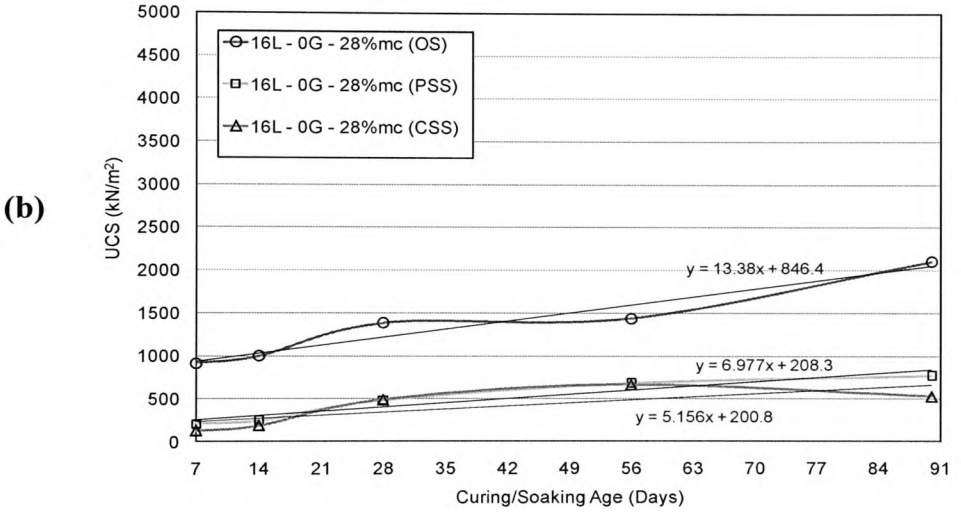
compaction moisture content than at high compaction moisture content. Apart from the improvement in strength all the other trends were repeated at all the compaction moisture contents.

Flash flood scenarios could last up to four days but where flood waters stay more than 4 days the likely destruction might be huge, hence, the need for an extended testing of the novel formulations. A 10-days soaking regime was followed and at the end the samples were tested for soaked Unconfined Compressive Strength (UCS). Fig. 5.37 shows the effect of 10-days soaking on the strength development of LOC stabilized with varying blended mix compositions after different moist curing ages. The strength development of the 16%Lime-0%GGBS at the various compaction moisture contents were very low as shown in Fig. 5.37a. However, the trend of strength development increasing with moist curing age was still observed similar to those of 4-days soaking period. On replacement of lime with GGBS at a ratio of 1:1 (8%Lime-8%GGBS) as shown in Fig. 5.37b the strength development was enhanced significantly. However, strength development was observed to develop with extended moist curing and soaking periods. It was also observed that strength increased with compaction moisture content to attain an optimum value at 28% compaction moisture content then reduced on further increase of moisture content (see Fig. 5.37b and c). Strength development continued gradually to attain the highest value at 56 days of moist curing and subsequent soaking for 10 days. The soaked compressive strength was observed to reduce after 56 days and this trend was observed with all the blended mixtures. Reducing the percentage of lime in the binder composition further to 4%Lime-12%GGBS still improved the strength development more. However, all the other trends that were identified earlier were also observed to have repeated. Comparing the results with those of 4 days soaking showed increased loss in compressive strength highlighting the effect of extended subjection of samples to harsh environmental pressure.

Fig. 5.38 shows the effect of stabilizing LOC with the different blended binders at specific compaction moisture contents. This further illustrates how lime replacement could affect the development of compressive strength. The highest compressive strength was observed with the samples compacted at 28% compaction moisture content as illustrated by Fig. 5.38b. From

Fig. 5.39 where the ultimate bearing strengths after 90 days moist curing and 10 days complete submersion results are shown, at 23% compaction moisture content the 16%Lime-0%GGBS showed compressive strength of about 200 kN/m^2. Reducing the lime content by 50% resulted in an overall increase of 1400 kN/m^2 and this was further enhanced when the lime content was reduced to a lime is to GGBS ratio of 1:3 (4%Lime-12%GGBS) a further increase in strength was observed. Increasing the compaction moisture content to 28% increased the bearing strength of the blended binders with the lime system forging a strength of about 380 kN/m^2 while the Lime-GGBS system produced compressive strengths of about 1700 kN/m^2 and 1900 kN/m^2 by the 8%Lime-8%GGBS and 4%Lime-12%GGBS blended binders respectively. Increasing the moisture content further to 33% however reduced the strength development though the effect of lime replacement was still observed.





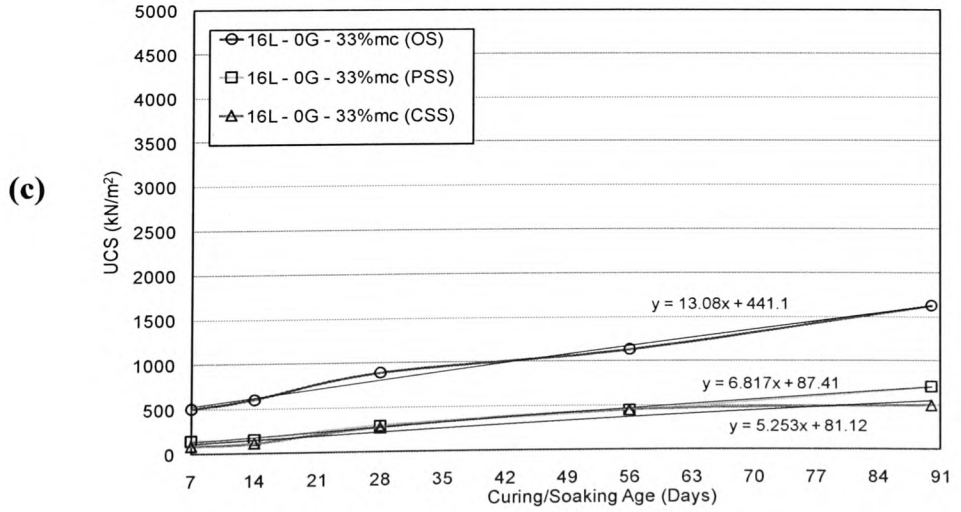


Fig. 5.33 Effect of 4-days soaking on the strength of 16%Lime-0%GGBS stabilized LOC (a) 23%m/c (b) 28%m/c (c) 33%m/c

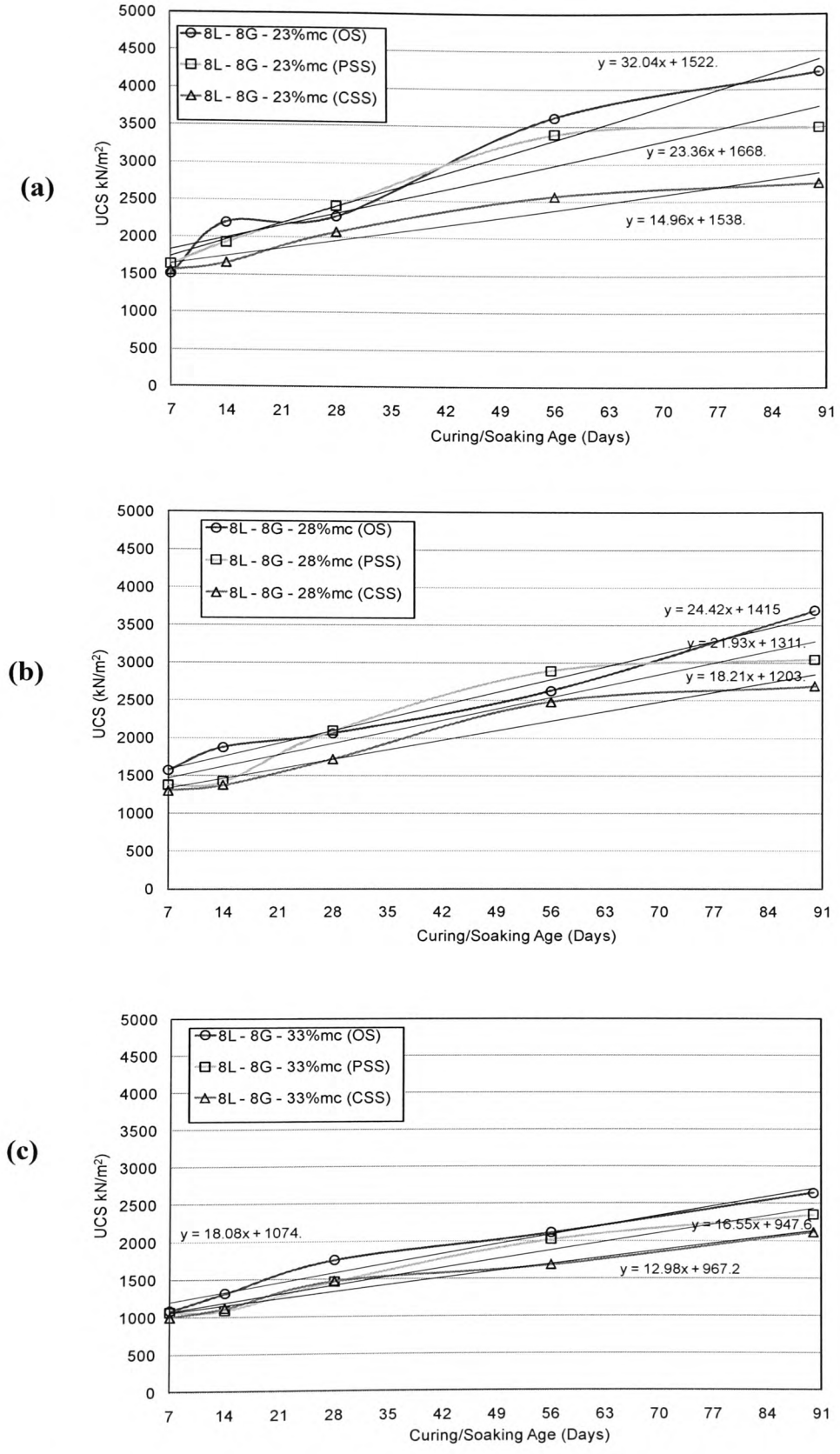
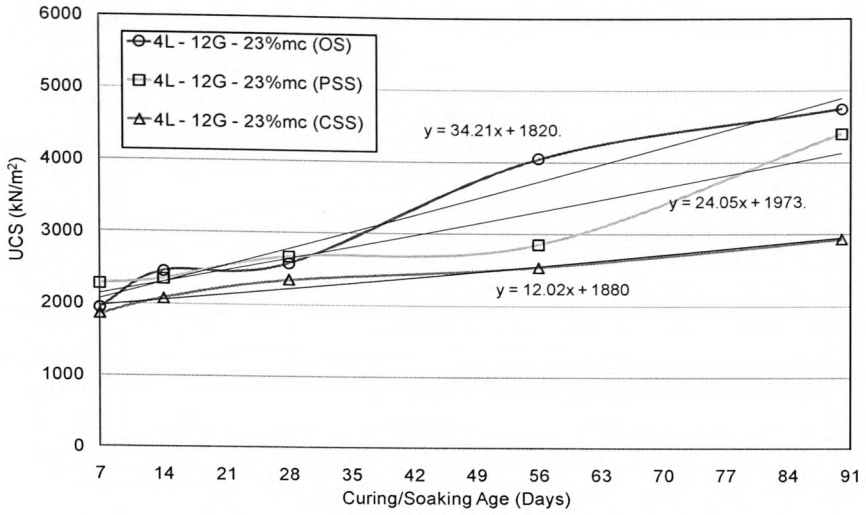
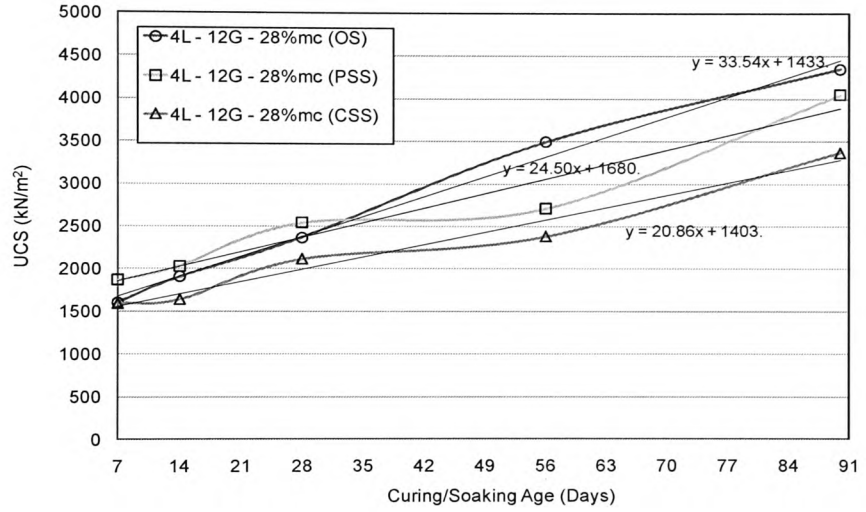


Fig. 5.34 Effect of 4-days soaking on the strength of 8%Lime-8%GGBS stabilized LOC (a) 23%m/c (b) 28%m/c (c) 33%m/c

(a)



(b)



(c)

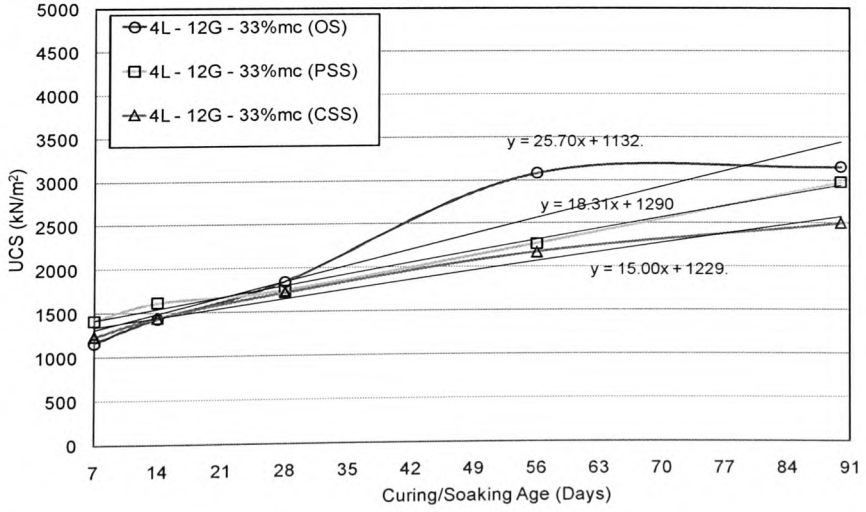
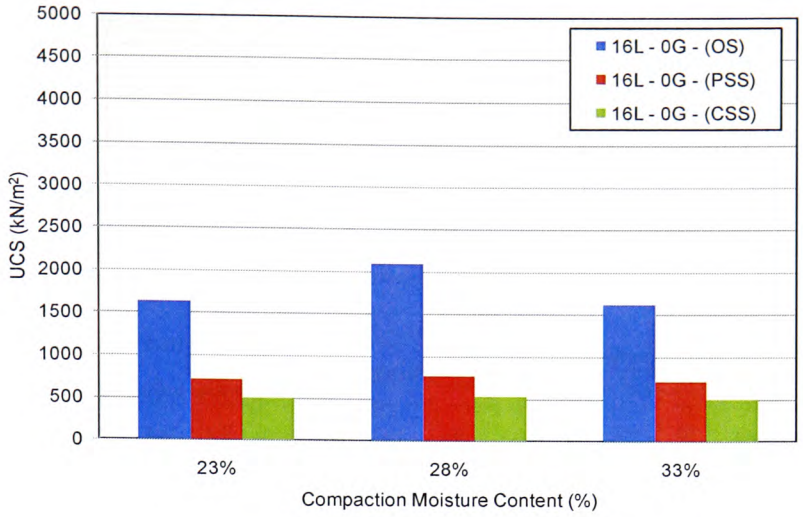
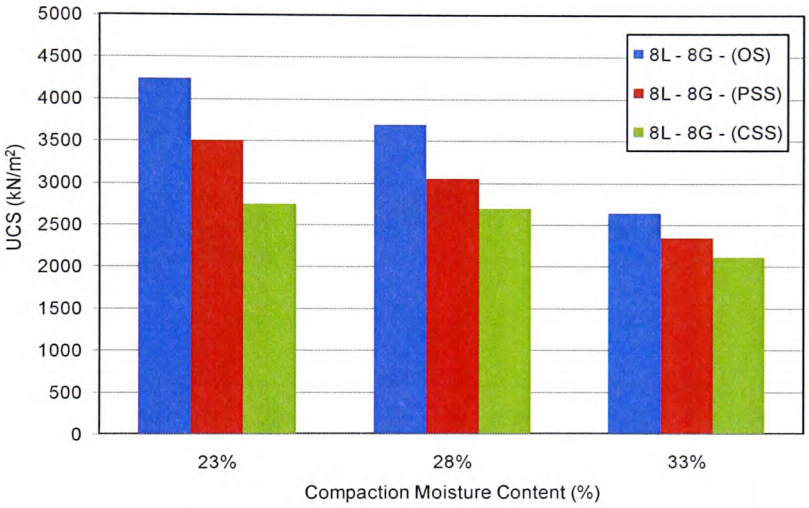


Fig. 5.35 Effect of 4-days soaking on the strength of 4%Lime-12GGBS stabilized LOC (a) 23%m/c (b) 28%m/c (c) 33%m/c

(a) - 16%L-0%G



(b) – 8%L-8%G



(c) – 4%L-12%G

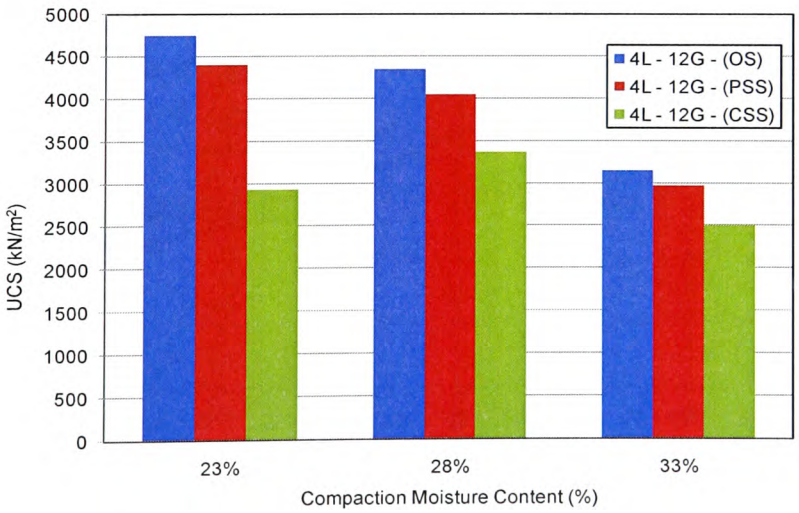
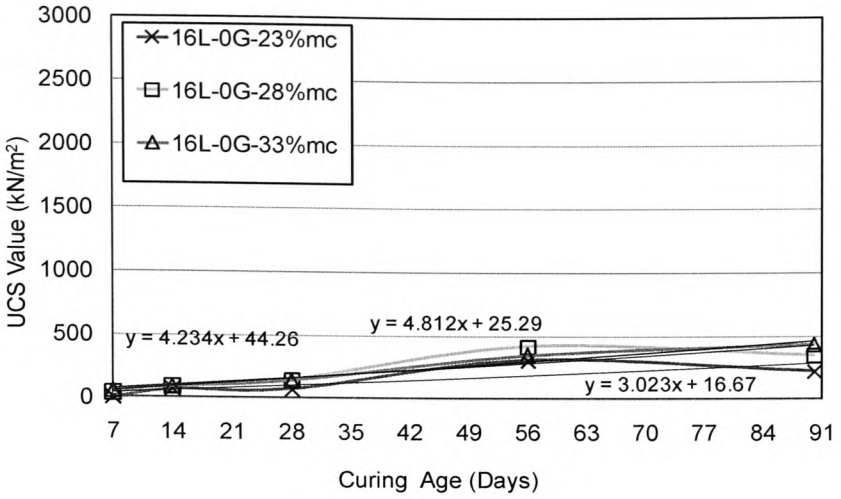
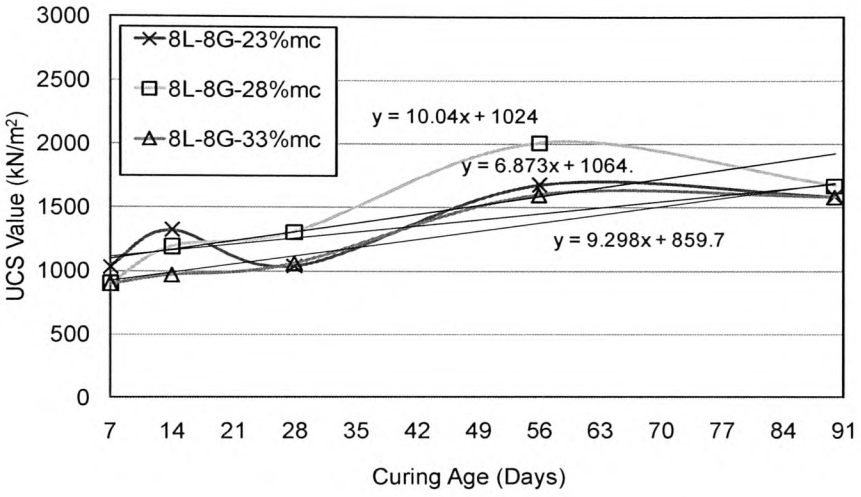


Fig. 5.36 Unconfined Compressive Strength of stabilised LOC using different blended binders after 90-days of moist curing and 4-days of partial and complete submersion in water compared with Unsoaked Strength of equivalent curing age.

(a)



(b)



(c)

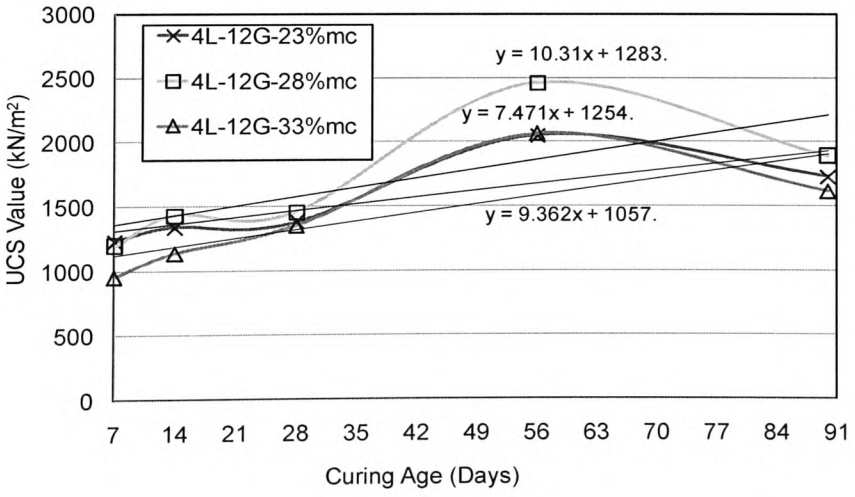
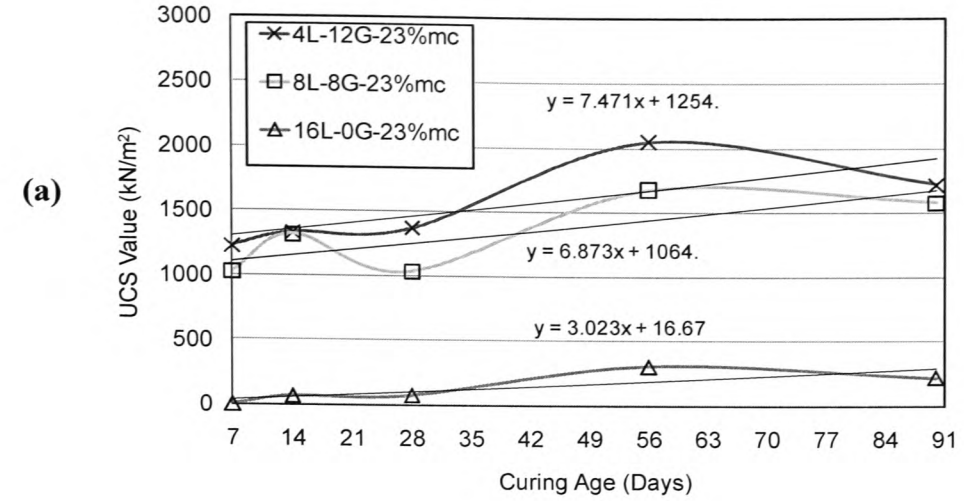
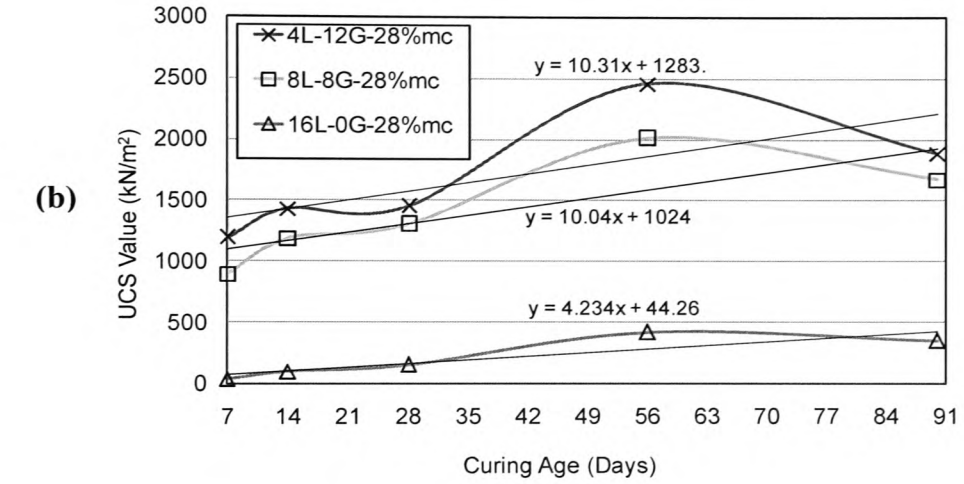


Fig. 5.37 Effect of 10-days soaking on the strength of LOC stabilized with varying blended mix compositions. (a) 16%Lime-0%GGBS (b) 8%Lime-8%GGBS (c) 4%Lime-12%GGBS





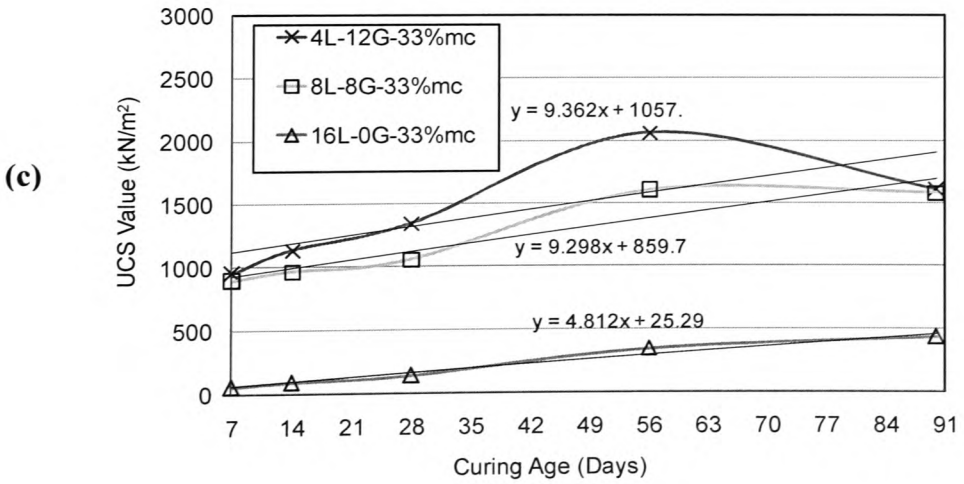


Fig. 5.38 The effect of 10-days soaking on the strength parameter of stabilized LOC at different moisture contents. (a) 23% m/c (b) 28% m/c (c) 33% m/c.

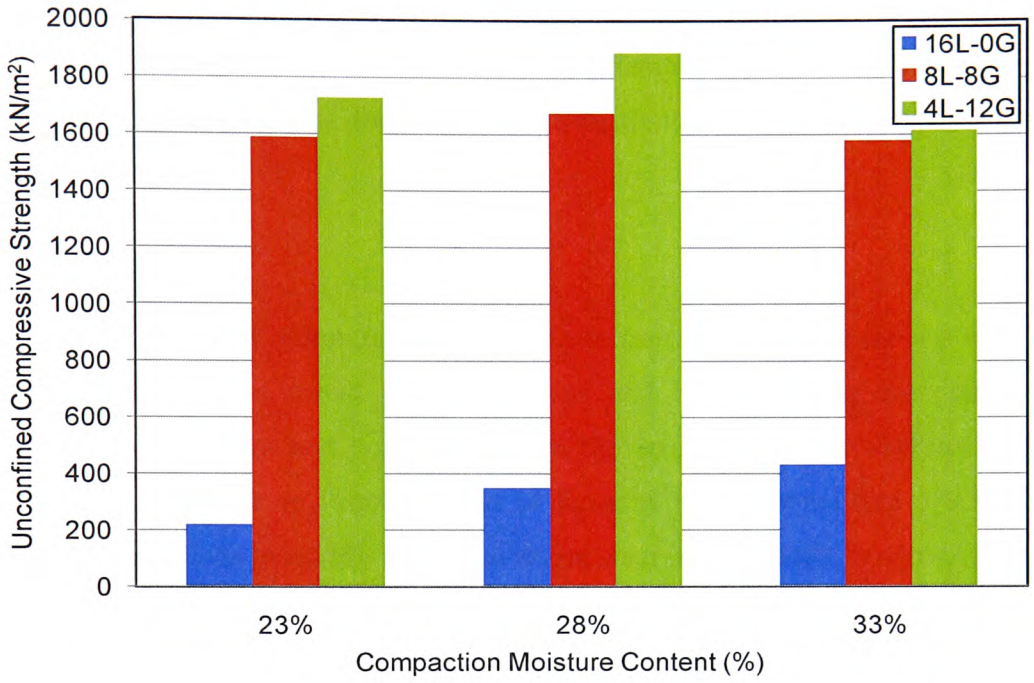


Fig. 5.39 Unconfined Compressive Strength of the different blends after 90-days moist curing then 10-days of complete submersion in water

5.4.3.2 Strength Reduction Indices (SRI)

Strength reduction index (SRI) or durability index (DI) is an empirical measure of how resistant a material is upon exposure to unfavourable environmental conditions. In the current research, the SRI is measured by obtaining the strength of the material after 4-days of partial and complete submersion in water or after an extended submersion period of 10-days and weighing them against the standard strength values obtained from the same samples in their dry state.

Fig.5.40 shows the result of submerging the different samples in water for a period of 4-days after different ages of moist curing samples and then carrying out compressive strength assessment on them and comparing with the strengths of dry specimens of the same composition. Without any replacement of lime as in the 16%Lime-0%GGBS the Strength Reduction Indices was very low as could be seen in Fig. 5.40a, b and c compacted at varying compaction moisture contents. With this system (lime system) at the different compaction moisture contents it is observed that the Strength Reduction Indices increased gradually with the age of moist curing to attain an optimum value after 56-days then reduced afterwards. In contrast, replacing lime with GGBS at different levels (1:1 and 1:3 blending ratios) offered a significant high Durability Index after 7 days then the indices reduced with moist curing/soaking age till the 90-days curing period. When lime is replaced with 50% of GGBS (see Fig. 5.40a, b and c) the Strength Reduction Indices was significantly improved. From Fig. 5.40a at 7 days over 90% of increase was achieved, however, upon extended curing /soaking the variation in Strength Reduction between the two systems (lime and lime-GGBS) was bridged to a large extend. When more lime was replaced by GGBS as in the 4%Lime-12%GGBS there was still a further enhancement in the Strength Reduction Indices though the trend of reduction with moist curing/soaking age was still prevalent. Another trend that was observed is that of increase in Strength Reduction Indices with increased compaction moisture content. From the results, it is evident that increasing the sample compaction moisture content improved the compaction characteristic and thereby reduced the detrimental effect of the aggressive environment on the samples, hence, better durability index at higher moisture contents. When the samples were stabilized and compacted at low moisture content, the tendency is that more water is absorbed upon soaking in water to satisfy the affinity for water by the lime component.

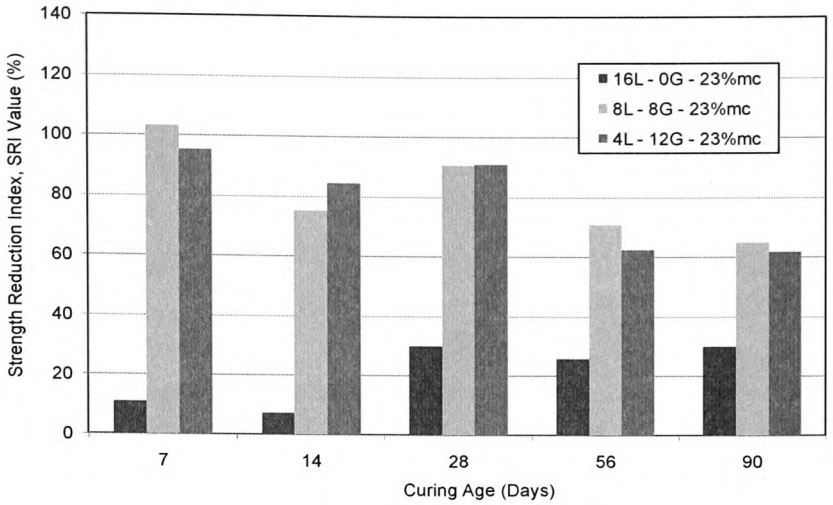
Presenting the results according to the different binders at the various compaction moisture contents as shown in Fig. 5.41 shows the vulnerability of the lime stabilised materials to soaking. Fig. 5.41a shows the Strength Reduction Indices of LOC stabilised with 16%Lime-0%GGBS at different compaction moisture contents. The results show very low Durability Indices at 7 days but a trend of increasing with moist curing/soaking period to attain the highest value after 56-days and then reduces till 90-days. It could also be observed that at 28% compaction moisture content the Durability Index (DI) was generally better. However, with the lime replaced samples the DI reduced with moist curing/soaking age and was better at higher compaction moisture contents.

Fig. 5.42 and 5.43 shows the effects on the Strength for partially soaking samples in water for 4-days and assessing their Strength Reduction Indices. It could be observed that reducing the harshness of the environment (effect of Soaking) improved the Durability Indices of the various samples (compare Fig. 5.42 and 5.43 with 5.40 and 5.41). Partial soaking ameliorated the harsh effect offered by the soaking environment; hence, better durability indices were obtained. The trend of better Strength Reduction Indices with increasing compaction moisture content also crystallised better from the results. Similar to the regime of 4-days complete submersion in water presented earlier, the 4-days partially submerged samples also followed the same trends that were already identified as could be observed in Fig 5.42 and 5.43.

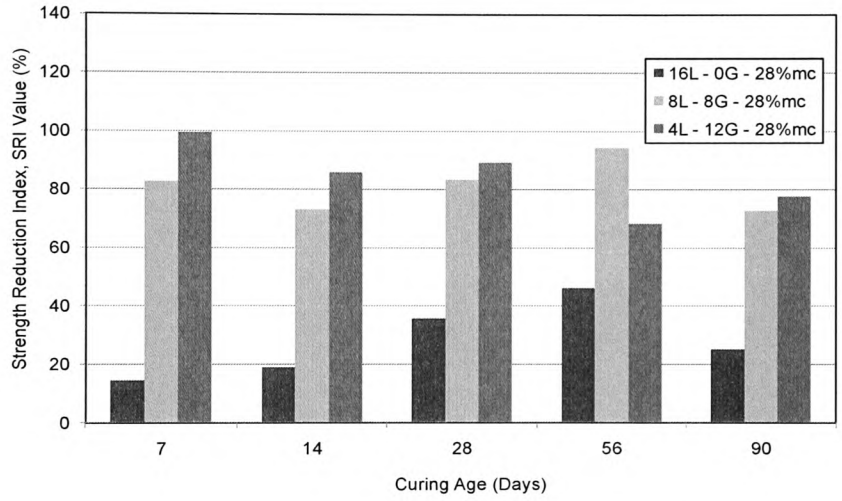
It is important to understand what is happening at the early stages of strength development upon flooding. This is why the current research happens to assess the 7- and 14-days Strength Reduction Index profiles of the stabilized material. At these early ages of strength development the effect of water ingress could be deleterious. The high risk of material failure associated with this early exposure could even be more dangerous when the submersion period is extended. Therefore, a rigorous experimentation was undertaken to ascertain the effect of complete submersion of the various stabilised samples in water for an extended period of 10-days on the strength parameter (see Fig. 5.44 and 5.45).

Fig. 5.44a shows the effect of 10-days complete submersion in water on the strength development of LOC stabilised with 16%Lime-0%GGBS at the various compaction moisture contents. Although the trend of increasing Strength Reduction Index with moist curing/soaking age is still observed it is also obvious how much increased environmental hazard could compromise strength development. The samples are also observed to perform better at increased compaction moisture contents. The lime replacement of lime at 8%Lime-8%GGBS is observed to improve Strength Reduction Indices even though the effect of extended submersion is still evident (see Fig. 5.44b). This improvement believed to have accrued from the addition of GGBS is even enhanced when more GGBS was introduced as in the 4%Lime-12%GGBS binder composition (see Fig. 5.44c). However, the trends observed with the 4-days of partial and complete submersion are still manifest. The trends illustrated by the results could be summarized as: reducing lime composition improves Strength Reduction Index; increasing moist curing/submersion age reduces Strength Reduction Index; increasing the compaction moisture content improves the Strength Reduction Index and Reducing the severity of the flooding (in terms of partial and complete submersion) also offers significant effect in the strength development and reduction characteristics.

(a)



(b)



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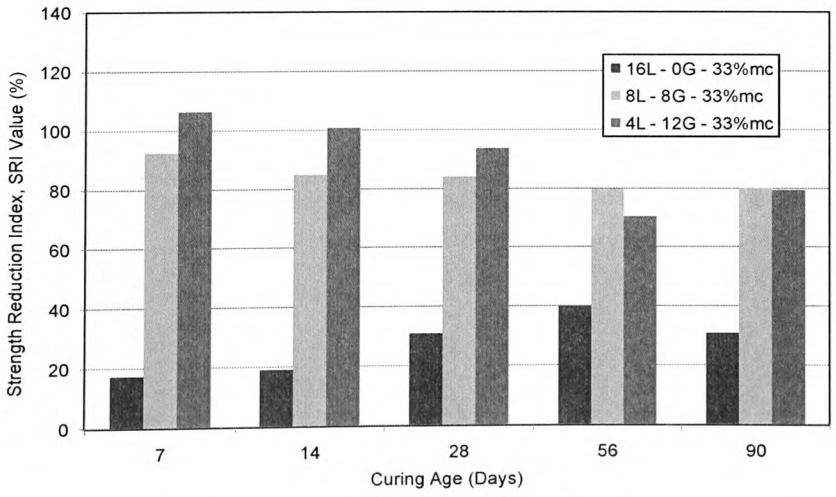


Fig. 5.40 Strength reduction index values of LOC stabilized with blended stabilizers and completely soaked in water for 4-days. (a) 23% m/c (b) 28% m/c (c) 33% m/c

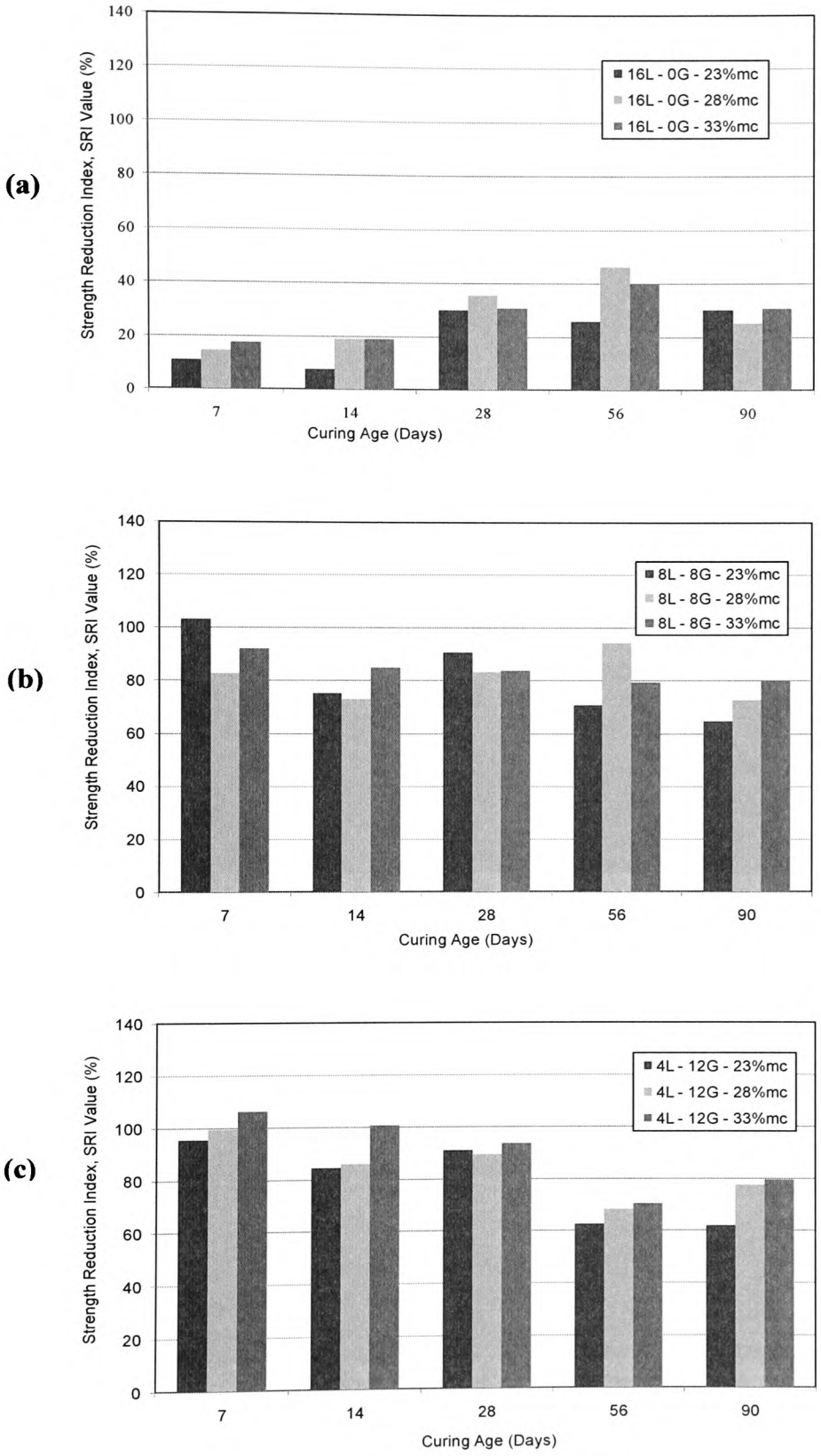


Fig. 5.41 Strength reduction index values of LOC stabilized with blended stabilizers and completely soaked in water for 4-days. (a) 4%Lime-12%GGBS (b) 8%Lime-8%GGBS (c) 16%Lime-0%GGBS

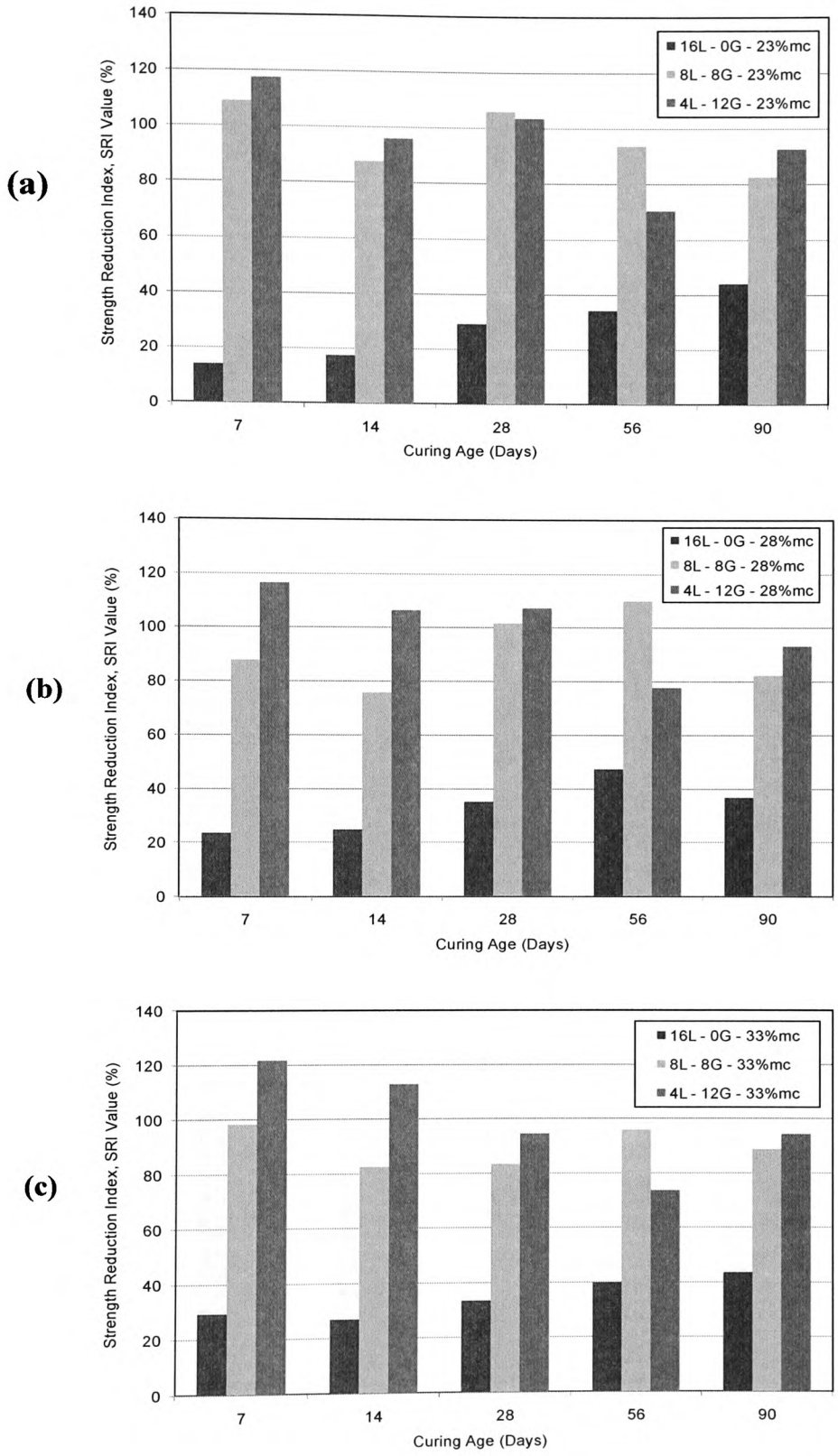
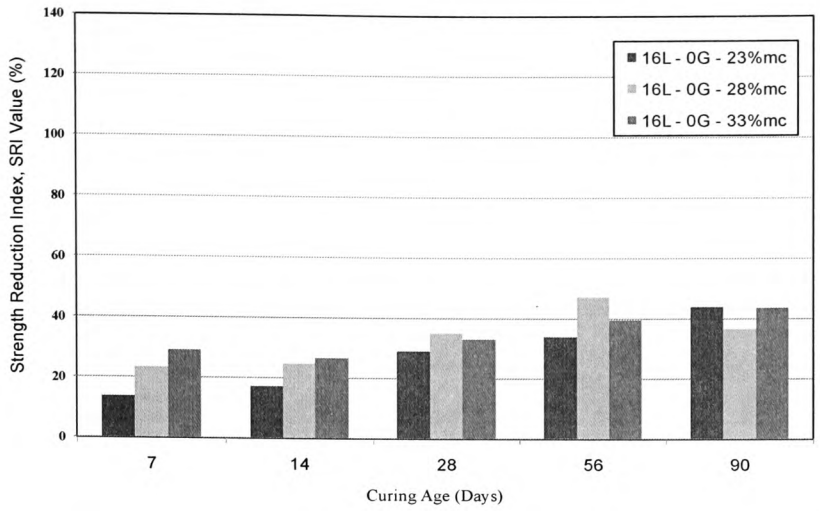
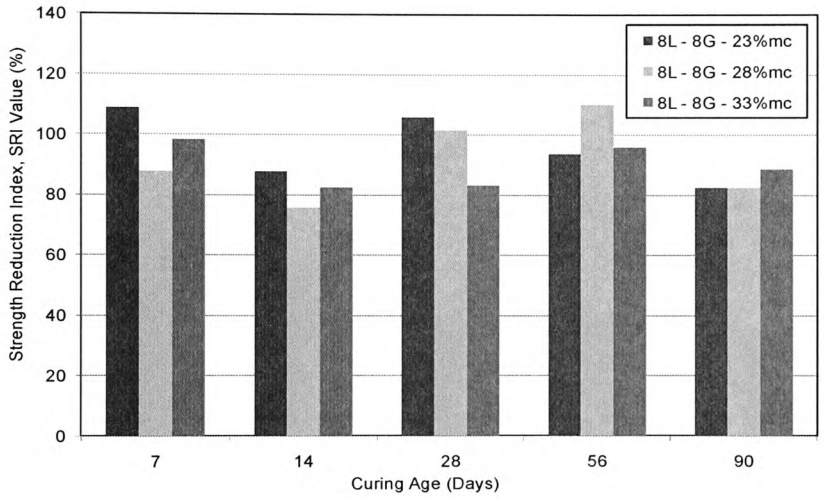


Fig. 5.42 Strength reduction index values of LOC stabilized with blended stabilizers and partially soaked in water for 4-days. (a) 23% m/c (b) 28% m/c (c) 33% m/c

(a)



(b)



(c)

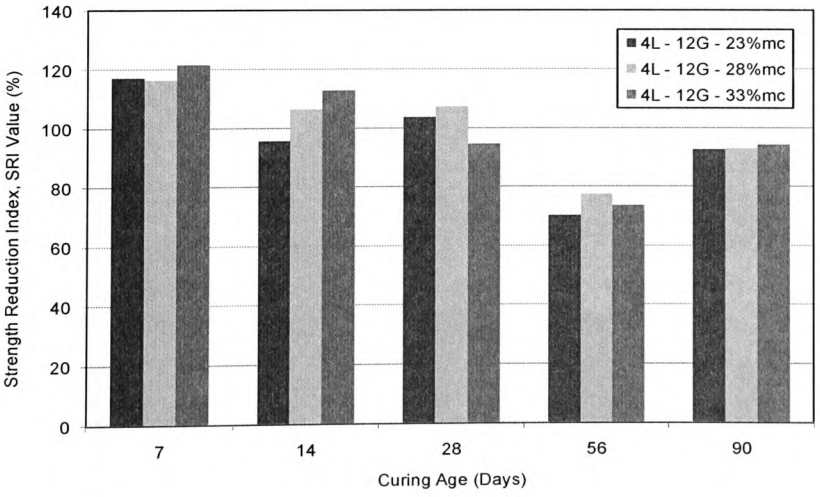


Fig. 5.43 Strength reduction index values of LOC stabilized with blended stabilizers and partially soaked in water for 4-days. (a) 4%Lime-12%GGBS (b) 8%Lime-8%GGBS (c) 16%Lime-0%GGBS

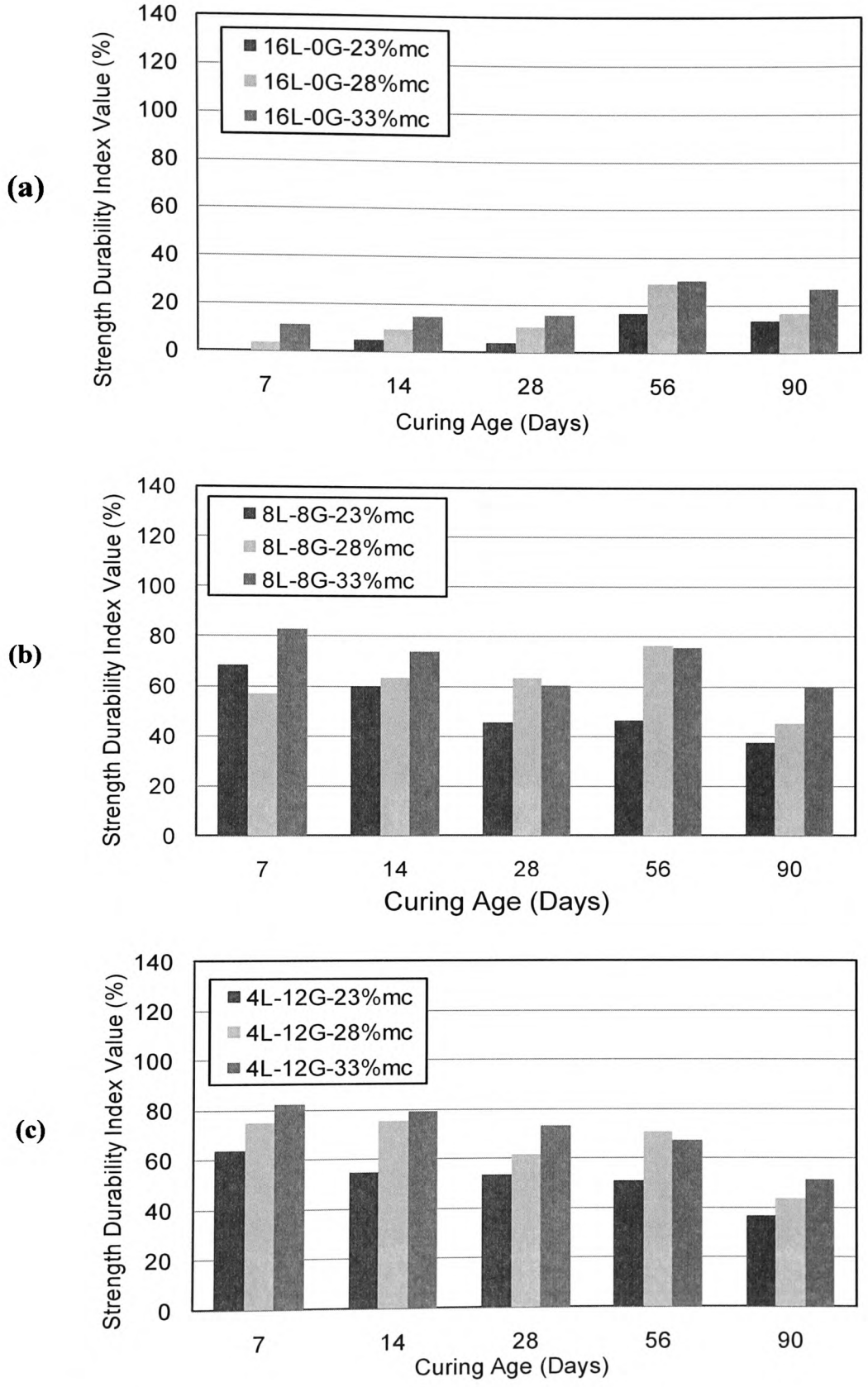


Fig. 5.44 Strength reduction index values of stabilized LOC completely soaked in water for 10-days. (a) 16%Lime-0%GGBS (b) 8%Lime-8%GGBS (c) 4%Lime-12%GGBS

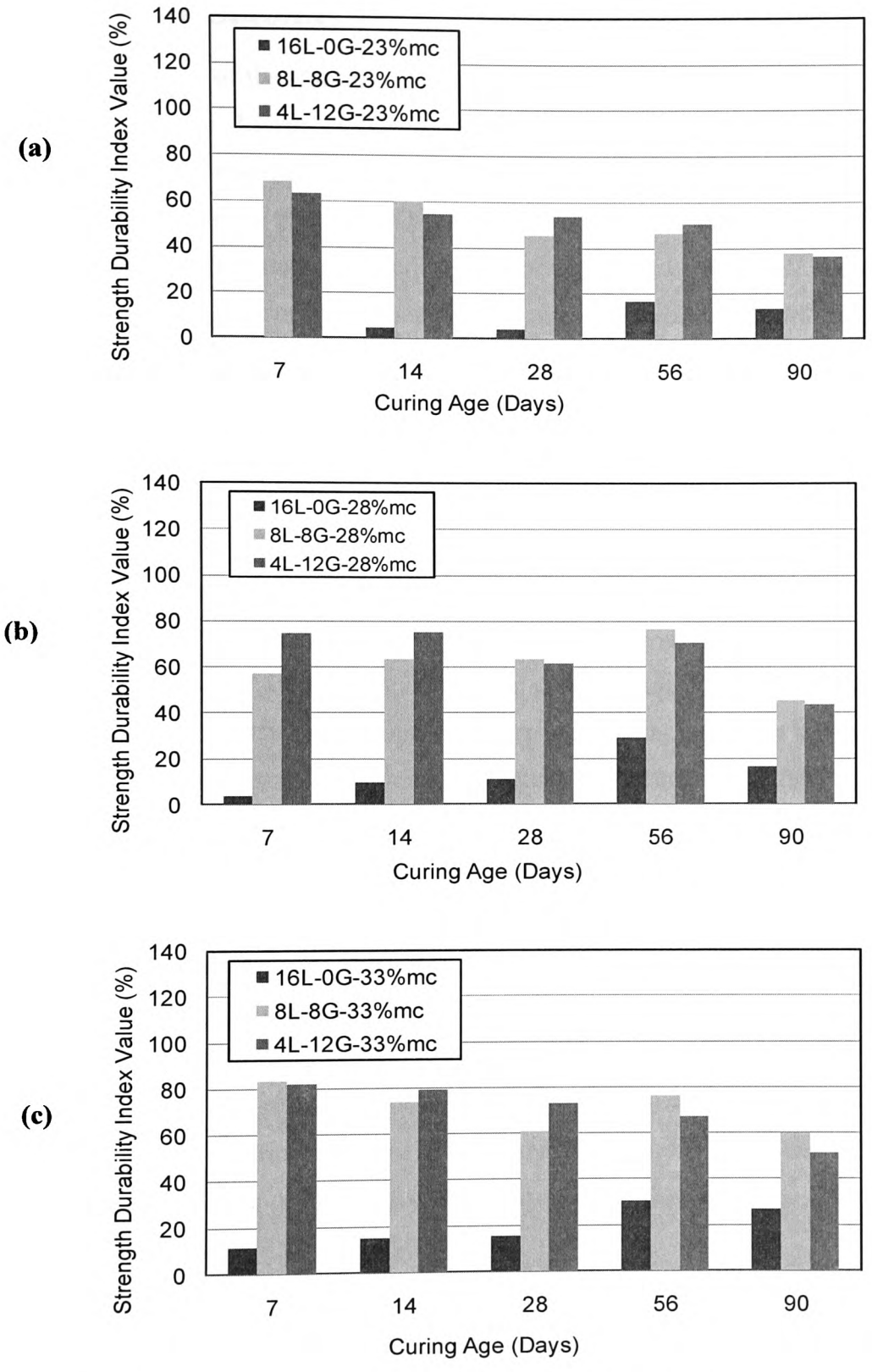


Fig. 5.45 Strength reduction index values of stabilized LOC completely soaked in water for 10-days. (a) 23% m/c (b) 28% m/c (c) 33% m/c

5.4.3.3 Volume Stability

The Volume Stability of the stabilized mixtures was ascertained through Linear Expansion measurements. These Linear Expansion (LE) test were carried out on three mix compositions of 4%Lime-12%GGBS, 8%Lime-8%GGBS and 16%Lime-0%GGBS with 16%Lime-0%GGBS as a control. LOC was stabilized with each of the blended mix compositions at four different compaction moisture contents of 23, 28, 33 and 38%. This investigation was carried out in two separate regimes – unmellowed and mellowed. Fig. 5.46 shows the results of the Linear Expansion measurement carried out on LOC stabilized with varying dosages of blended stabilizer at different compaction moisture contents. Fig. 5.46a is the volume stability assessment of 16%Lime 0%GGBS moist cured for 7 days and soaked thereafter for up to 50 days. Without any lime replacement high Linear Expansion values were observed. The expansion started from the start of the test and increased progressively throughout the 50 days observation period. This otherwise means increase in Linear Expansion with curing/soaking age. It is also observed that the rate or volume of expansion was dependant on the compaction moisture content of the various samples. The samples compacted at 23% compaction moisture content showed high values of volume instability and as the compaction moisture content was increase to 28% the expansion reduced. This reduction in expansion with increase in compaction moisture content continued when it was further increased to 33% and finally to a moisture content of 38%. Upon the replacement of lime with GGBS to arrive at 8%Lime-8%GGBS (Fig. 5.46b), the Linear Expansion was reduced more than a half. However, the trend of expansion due to compaction moisture content variation still continued. It is also observed that when lime was replaced with GGBS, the expansion was within the moist curing period after which time the volume of the samples appeared to be stable throughout the soaking period. Further replacement of Lime to attain a 1:3 Lime:GGBS ratio (4%Lime-12%GGBS) as shown in Fig. 5.46c did not appear to bring about much change in the volume stability measurement.

Fig. 5.47 compares the different blended mixtures at different compaction moisture contents after the ultimate moist curing/soaking age of 50 days. From the results it is observed that replacing lime with GGBS brought about a significant reduction in the expansion characteristics of the stabilized material. This reduction in the expansive

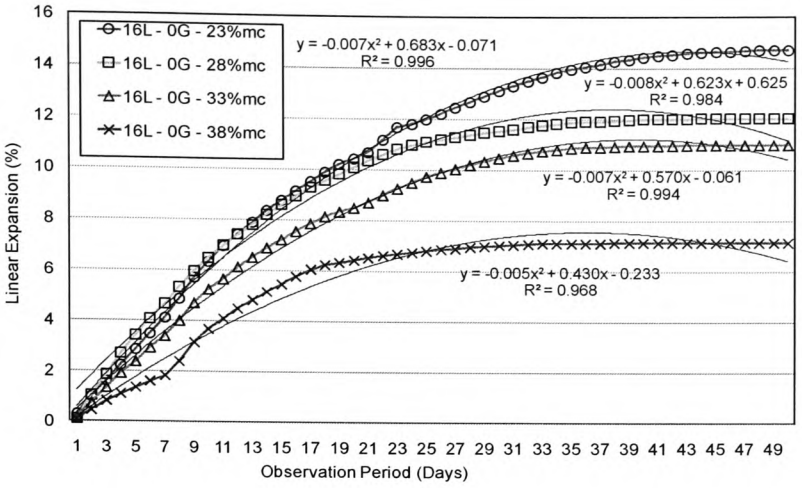
behaviour of the stabilized material continued though on a reduced scale when the lime content was further reduced to attain the 4%Lime-12%GGBS blended mix composition. From Fig. 5.47 at no lime replacement and 23% compaction moisture content a Linear Expansion of 15% was observed. This high Linear Expansion behaviour could represent significant volume instability when lime is applied to expansive clay soils. On increasing the compaction moisture content to 28% the Linear Expansion reduced to 12% representing a 25% reduction in the expansion characteristics. Further increase in the compaction moisture content to 33% still resulted in a further reduction to 11% and finally to 7% upon increasing the compaction moisture content to 38%. However, replacing lime with GGBS at a level of 50% (8%Lime-8%GGBS) (see Fig. 5.47) and at 23% compaction moisture content showed about 4% expansion, on increasing the compaction moisture content further to the last level of 38% the expansion was observed at about 1%. The effect of further replacement of lime to attain 4%Lime-12%GGBS on the expansion characteristics was not significant.

Lime offers the property of premixing with clay before compaction, an exclusive attribute that reduces the plasticity of the stabilized material (clay) and increases overall workability. The process of mellowing involves the mixing of the soil with a given percentage of moisture and lime, and concealing the material mixture in an environment controlled chamber. This could cure for up to 72 hours within which time the clay minerals are able to engage the lime resulting in characteristic property changes that enhances the overall workability of the stabilized material. On successful completion of the mellowing period, the material is then remixed with the remaining percentage of the water and lime and compacted to the required specification. This has been appraised by other researchers to reduce the linear expansion of expansive clays. Fig. 5.48 and 5.49 shows the effect of mellowing on the linear expansion of stabilized LOC at different compaction moisture contents and varying stabilizer blends. From the results mellowing do not seem to have much impact on the expansive behaviour of the various samples. From Fig. 5.48a it is observed that at high level of lime stabilization 16%Lime-0%GGBS the expansive behaviour was high though the influence of the mellowing was present but not significant. The expansion behaviour is noticed throughout the periods of moist curing and soaking. It is also observed that the expansive behaviour reduced with increase in the compaction

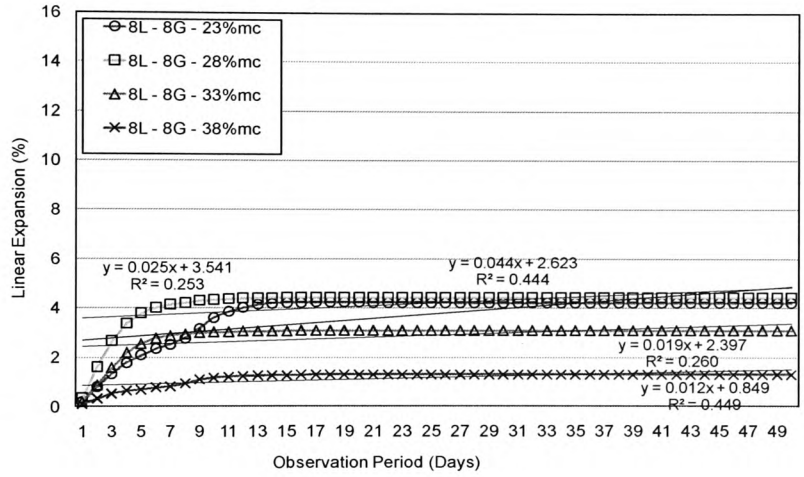
moisture content of samples. Replacing the lime component of the mix with 50% GGBS (see Fig. 5.48b) had a noticeable influence on the expansive behaviour, however, with the lime replaced samples the expansion was only within the period of moist curing within which time the GGBS is believed to have been activated to resist any further expansion. It is also observed that the expansion reduced with an increase in the compaction moisture content at which samples were produced. For the mellowed samples, further replacement of lime to attain the 4%Lime-12%GGBS as shown in Fig. 5.48c illustrated a further reduction in expansive behaviour. As with the other mix compositions, the rate of expansion reduced with increased compaction moisture content.

From Fig. 5.49 that shows the Linear Expansion behaviour of stabilized LOC after 3 days of mellowing, it is observed that when lime was not replaced the expansion was high. The 16%Lime-0%GGBS stabilized LOC at 23% compaction moisture content after the end of the observation period showed an expansion value slightly above 14% (see Fig. 5.49) as against 15% of the unmellowed samples as observed in Fig.5.47. Increasing the compaction moisture content to 28% reduced the expansion to about 11% and further increase to 33% reduced it even lower to 10%. For the 8%Lime-8%GGBS blended mix composition at 23% compaction moisture content the expansion was 6%, reducing to 5% on increasing the compaction moisture content to 28% and then to 4% at 33% compaction moisture content. Similarly, the 4%Lime-12%GGBS stabilized LOC at 23% moisture content expanded to 4% after 50 days of moist curing/soaking and reduced to 2% on increasing the compaction moisture content to 28% and a further reduction to 1% when the compaction moisture content was reduced to 33%.

(a)



(b)



(c)

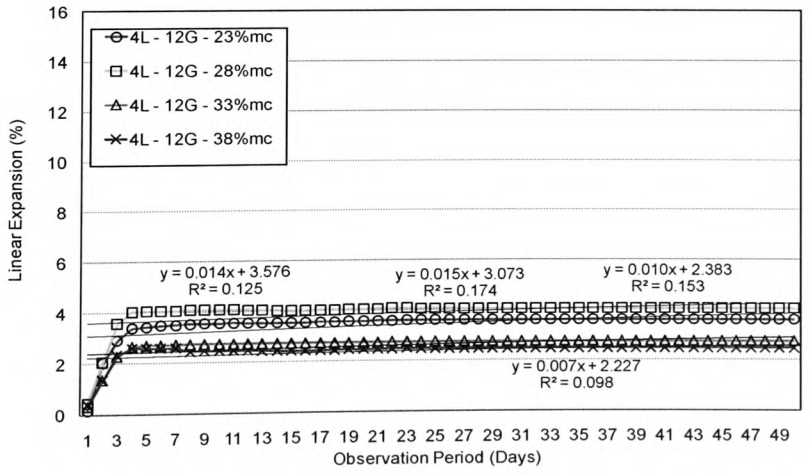


Fig. 5.46 Linear Expansion of LOC stabilized with varying blended stabilizer at various levels of moisture content additions. (a) 4Lime-12GGBS, (b) 8Lime-8GGBS, (c) 16Lime-0GGBS

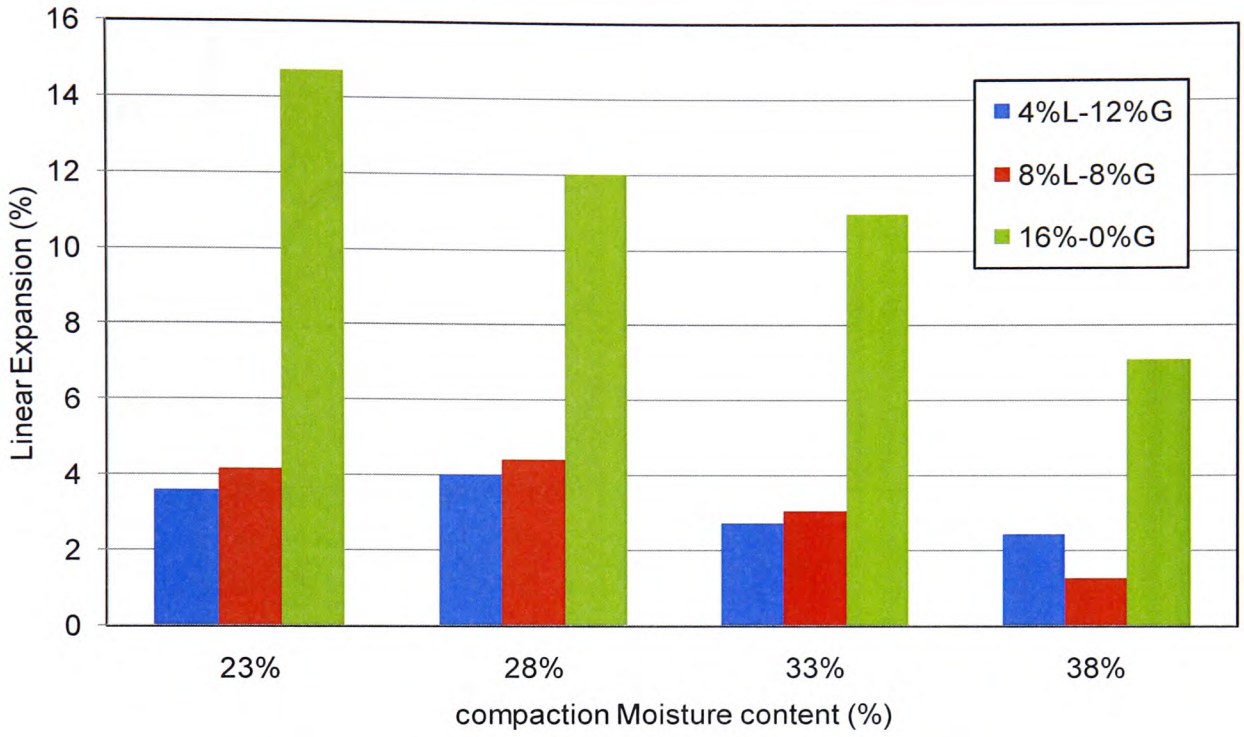
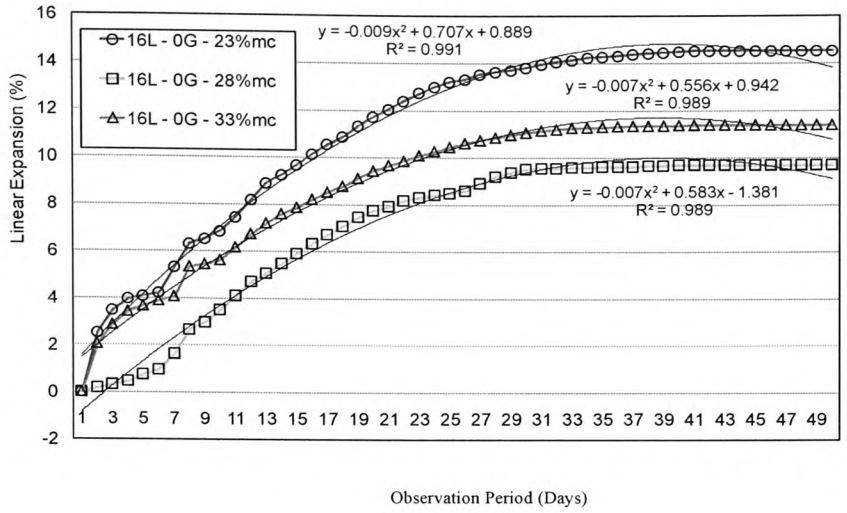
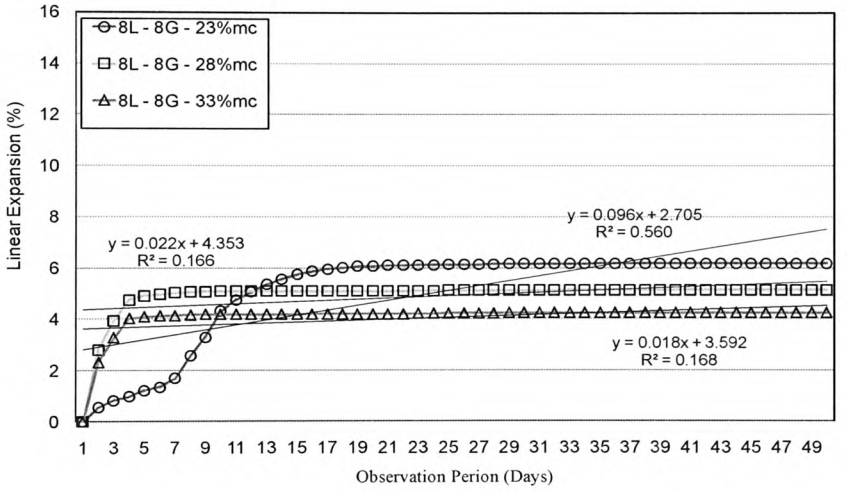


Fig. 5.47 Linear Expansion of LOC stabilized with different stabilizer blended mixes at varying moisture content.

(a)



(b)



(c)

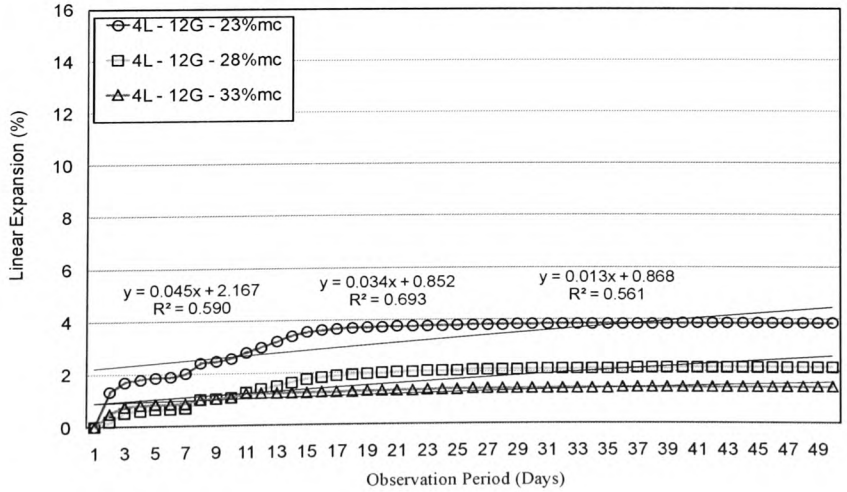


Fig. 5.48 Linear Expansion of stabilized LOC after 3-days mellowing. (a) 4Lime-12GGBS (b) 8%Lime-8%GGBS (c) 16%Lime-0%GGBS

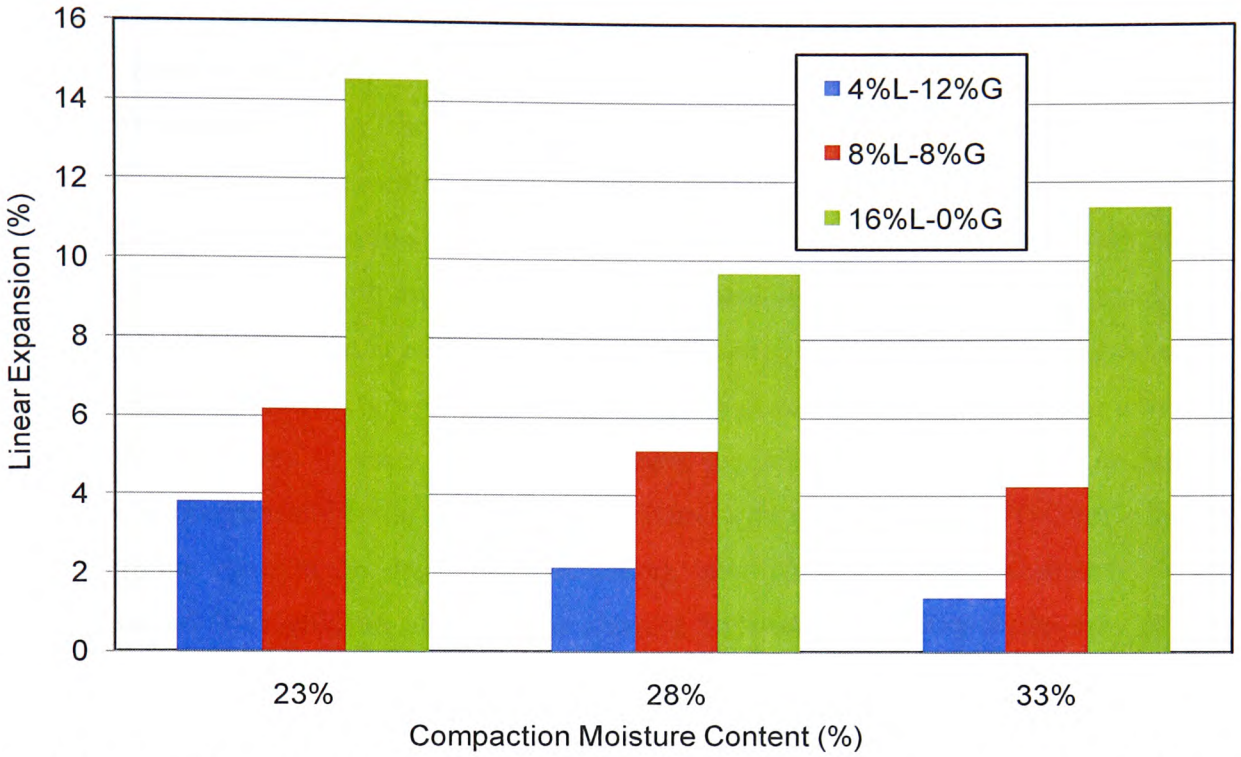


Fig. 5.49 Linear Expansion of stabilized LOC after 3-days mellowing a comparison of the different blended mixes at varying compaction moisture content.

5.4.3.4 Compressibility

The dimensional stability of stabilized LOC using varying blended mixes at different curing time and compaction moisture contents has been assessed using the linear expansion parameter (see Section 5.5). However, this section examines and compares the strain suffered by samples upon the application of stress. This compression in practical applications could be perceived as either settlement or consolidation due to loading. Table 5.4 illustrates the compression on different soak lime/GGBS stabilized materials after varying ages of moist curing and 10 days complete soaking in water. From the result in Table 5.4 it is observed that at each particular age of curing the samples followed the trend of decreasing compressibility with increasing compaction moisture content. This is a trend that was also observed in section 5.5 on the linear expansion behaviour of the different samples. The compressibility is also seen to decrease with age from the early curing ages to an optimum value at 56 days then decreasing after wards as curing continues. The highest level of lime addition of 16% lime showed high compressibility at the very early ages of curing. The strain upon loading reduced with time as the samples developed more strength to cushion the effect of the applied load. With a 7 days compression of 4.5%, 4.5% and 4% respectively for the 16%Lime-0%GGBS at 23% m/c, 16%Lime-0%GGBS at 28% m/c and 16%Lime-0%GGBS at 33% m/c an improvement of 1.5%, 2% and 2% was achieved after 56 days curing for the 16%Lime-0%GGBS at 23% m/c, 16%Lime-0%GGBS at 28% m/c and 16%Lime-0%GGBS at 33% m/c respectively. However, on the replacement of 75% and 50% lime with GGBS (4%Lime-12%GGBS and 8%Lime-8%GGBS) an improvement of 0.5%, 0.5% and 1% was achieved after 56 days curing of 23%, 28% and 33% compaction moisture contents respectively.

Table 5.4 compression of samples as a percentage of length of sample after different curing periods and 10-days complete soaking in water.

S/N	Sample	7 days Ave Extension (%)	14 days Ave Extension (%)	28 days Ave Extension (%)	56 days Ave Extension (%)	90 days Ave Extension (%)	150 days Ave Extension (%)
1	4L-12G-23%mc	3	3	2.5	2.5	2.5	3
2	4L-12G-28%mc	2.5	2.5	2	2	2	2
3	4L-12G-33%mc	2	1.5	1.5	1.5	1.5	2

S/N	Sample	7 days Ave Extension (%)	14 days Ave Extension (%)	28 days Ave Extension (%)	56 days Ave Extension (%)	90 days Ave Extension (%)	150 days Ave Extension (%)
4	8L-8G-23%mc	3	3	2.5	2.5	2.5	3
5	8L-8G-28%mc	2.5	2.5	2	2	2	2
6	8L-8G-33%mc	2	1.5	1.5	1.5	1.5	2

S/N	Sample	7 days Ave Extension (%)	14 days Ave Extension (%)	28 days Ave Extension (%)	56 days Ave Extension (%)	90 days Ave Extension (%)	150 days Ave Extension (%)
7	16L-0G-23%mc	4.5	4	4	3	3	3.5
8	16L-0G-28%mc	4.5	4	3.5	2.5	2.5	3
9	16L-0G-33%mc	4	4	3.5	2.5	2.5	3

Chapter Six: Discussion

The overall research project sets to distinguish two assessment criteria for determining the robustness of a lime and lime blended stabilizer for the stabilization of flood prone areas - those that border on the strength parameters and the others on durability. There is a relationship between strength and durability parameters of stabilized soil which is not yet very well understood. In general, high strength tends to imply good durability. Durability is particularly critical to road structural layer formations and this chapter is designed to explore the practical implications of stabilizing flood prone areas with lime blended with industrial waste materials. It is anticipated that better ways of strength enhancement and durability improvement will be achieved. Durability related problems are common place especially when lime is used to stabilize sulphate-bearing clay soils that are susceptible to flood inundation or are in contact with water. This has been due to the enhanced growth of expansive products in the presence of moisture. This has been the subject of several research projects in the recent past hence, the current work aims to go a step further by carrying out soil stabilization at a level above the classical dosage levels. It has the aim of curtailing the volume instability and loss of strength upon contact with water often suffered by stabilized soil materials.

6.1 Strength Parameters

The major assessment criteria used by the current research to appraise the compressive strength parameter of lime and Lime/GGBS stabilized expansive clay soils is by the direct measurement of bearing strength using the Unconfined Compressive Strength test method. In Britain, an unconfined compressive strength value of 1754 kN/m^2 after 7 days of curing is considered acceptable as reported by Stavridakis (2005). Hafez et al (2008) also reported that the strength of stabilized soil materials depends largely on the formation of the two cementing phases C-S-H and C-A-H gels and these may require the dissolution of silica and alumina from the soil components.

In a typical soil, the possible sources of silica and alumina are the clay minerals, quartz, feldspars and micas with the occurrence of other silicates and alumina silicates present

either as crystalline or amorphous phases. The solubilities of silica and alumina are both increased in the elevated pH conditions that prevail in the Portland cement and lime-soil systems which results in the formation of calcium-silicate-hydrate (C-S-H) that strengthens a soil that is stabilized with lime or with lime activated cementitious substances through pozzolanic and hydration processes, whereby silica and alumina are dissolved in the high pH environment to enable the reaction with calcium and hydroxyl ions from the lime to take place.

A pozzolanic reaction is a slower process relative to the hydration reaction involving lime activated slag. Pozzolanic reaction produces materials with a cementitious property, expressed chemically thus:

$$Ca(OH)_2 + SiO_2 \rightarrow C\text{-}S\text{-}H$$

$$Ca(OH)_2 + Al_2O_3 \rightarrow C\text{-}A\text{-}H$$

The strength developed by a stabilized material is influenced by the quantity of cementitious gel produced and consequently on the amount of lime consumed. Extended curing times and elevated temperatures are some of the ingredients that facilitate pozzolanic reactions. By extending the curing time the growth of reaction products are enhanced which will obviously increase the bearing capacity of the stabilized material. This is corroborated by the findings of Bell (1996). From Figures 5.3. – 5.7 it is evident that extended curing times enhanced strength development while Figures 5.8 and 5.9 also showed high strength values after 7 days of moist curing. Although the strength of test specimens in general increased with the addition of stabilizer, however, on adding lime more than the threshold value of 4%, there was no significant increase in strength (see Fig. 5.6).

On the other hand, when lime was gradually replaced by GGBS the strength increased (see Fig. 5.6) until a limiting value was reached after 75% lime replacement. According to Tasong et al (1999) it is owing to the free lime available in the system; the lime in a Lime-Clay-GGBS system could be reduced by two processes - through the gradual replacement

of lime by GGBS as a result of dilution and secondly by the GGBS hydration process which improves cementation properties and ultimately depresses pH. Figures 5.3(a) and (c) compare lime and Lime/GGBS blended mixes; it is seen that the addition of GGBS facilitated the gain in strength. This is believed to have stemmed from the increased production of cementitious products facilitated by the addition of GGBS. The lime component of the lime-Clay-GGBS system ensured the high pH medium requirement, that is necessary to activate GGBS and the continued presence of Ca^{2+} ions from the lime was useful in maintaining the pozzolanic reactions that enhanced strength gain.

Due to the differences in mix proportioning, the rate of reaction of the different blends are different. This explains why the variations in the results of the different blended compositions. From Fig. 5.3(c), 5.5 and 5.6 it is also clear that if there is no lime to activate the GGBS, not much could be expected from the GGBS in terms of strength gain even after long periods of curing. It has been reported by several investigators on stabilized expansive clay soils that the strength achieved by this type of material is not absolutely from C-S-H and C-A-H cementing products but could also be enhanced by ettringite formation (Wild et al, 1998; Kinuthia et al, 1999; Modmoltin and Voottipruex, 2009; Kamon and Nontananandh, 1991). In the early stages of monitoring volume stability there exists tendencies of expansion, however, within which time the GGBS swifts into action and arrested further growth of expansive compounds.

As shown in Figures 5.7, 5.8 and 5.9, the results of the 4%Lime-12%GGBS and 8%Lime-8%GGBS have been very competitive and this could also be due to the fact that up to 6% lime is still within the lime fixation point which has been reported by Hewayde et al (2005) as ranging from 4% - 6%. Alternatively, when a stabilizer is added in doses more than a given threshold, soft pockets are likely to be left on the material. These might result from unreacted stabilizer material which will subsequently become sources of weakness to the stabilized material (Rao et al, 2008). In the case of lime stabilizers, the unreacted materials are very likely to be engaged in carbonation reactions which convert the lime to a compound (calcium carbonate, $CaCO_3$) that is not useful for soil stabilization but rather has agricultural applications. This results from the following reversible process.

$$CaO + CO_2 \rightleftharpoons CaCO_3$$

The reversing of this process involves the application of a high temperature to expel the carbon dioxide compound with quicklime as the product of the reaction.

6.1.1 Effect of Stabilizer on Clay Soil

Lime stabilization has been used for many years to improve soil properties. When lime (CaO) is mixed with clay soil, the tendency is that the CaO engages the clay minerals in a cation exchange reaction. This results in the flocculation of fine particles of clay – a situation whereby the fine clay particles come together to form larger particles. This is otherwise known as lime modification, this possesses the attribute of improving the engineering characteristics of a lime treated soil.

A stronger agglomeration of fine particles in the clay soil usually follows the initial reaction and the plasticity of the soil may be reduced. This process is necessary to improve the workability of clay soils for civil engineering applications. The process involves the hydration reaction of quicklime; when quicklime is in contact with water it hydrates rapidly liberating much heat which keeps the system drier with improved strength. The reaction which is expressed chemically as illustrated below is also known as the slaking of lime (Puppala et al, 2003; Bell, 1996; Wild et al, 1996; Tasong et al, 1999; Perry et al 1996).

$$CaO + H_2O \rightarrow Ca(OH)_2 \quad \Delta H = -65.27 \text{ kJ/mol}$$

After this initial swift effect of lime on the clay, more permanent reaction processes are initiated. These are the pozzolanic and hydration reaction processes especially where the uses of latently cementitious materials are employed. The pozzolanic reactions are generally characterized by time dependence, the best performance being always expected from long cured specimens. Hydration reactions are much faster but are most effective at elevated temperatures of about 20°C.

The result of this second level reaction is responsible for strength development. This is usually through the formation of Calcium – Silicate – Hydrate (CSH) and Calcium – Aluminate – Silicate – Hydrate (CASH) gels from both the hydration and pozzolanic reactions. The formation of these cementing gels is critical to strength development. In addition to the physical effects of lime on stabilized clay, quicklime also promotes the solubility of SiO_2 and Al_2O_3 present in the clay to produce the high pH environment that supports further reaction and the production of more Calcium-Aluminium-Silicate-Hydrate (CASH) gel. This results in further enhancement of strength, reduction in pore spaces, volume stability, increased density and improved intrinsic permeability of the stabilized material. The exclusive quality of autogenous healing offered by lime stabilized soil materials is also attributable to the pozzolanic reaction of lime. This reaction process could last for years as in many recorded cases (Al-Rawas et al, 2005; Gallala et al, 2007; Barker et al, 2007; Hewayde et al, 2005; Bell, 1996; Wild et al, 1996; Prusinski and Bhattacharja, 1999; Puppala et al, 2003; Oti et al, 2008; etc.)

The best hydration reaction is possible at a given pH range and this range of pH tends to coincide with the optimum lime content required for a given stabilization. According to Hewayde et al (2005) this optimum pH range could be obtained at a lime dosage of about 4-6%. At this level of stabilization, the silica and alumina present in the clay particles are optimized as they dissolve more. The dissolved silica and alumina reacts with the Ca^{2+} present in the lime to form materials with a cementitious value. On increasing the lime content of the mix above this threshold, the strength parameter is observed not to increase (see Fig. 5.3; 5.5 and 5.7) and there is also the apprehension that volume instability will be encouraged (see Fig. 5.46 - 5.49). This, according to Manasseh and Olufemi (2008), could have serious durability implications. According to Bell (1996), the highly alkaline environment produced by the addition of lime gives rise to the slow solution of alumino-silicates which are then precipitated as hydrated cementitious reaction products. For maximum reactivity, the pH value of the pore fluids in the voids should remain at around 12.4 as the solubilities of silicon and aluminium ions are very high at this value. Hughes and Glendinning (2004) also stressed the importance of pH in soil stabilization as a means of optimizing strength development.

The blending of GGBS and lime in the stabilization of clay soil has been reported by several investigators as a contemporary technology with much benefit. Puppala et al (2003) recognised five factors as critical to strength enhancement in a GGBS activated soil stabilization process. These are chemical composition, the alkali concentration of the reacting system, the glass content of the slag, the fineness of the slag and the temperature during the early phase of the hydration process. Similarly, Wild et al (1999) reported that the addition of GGBS into the lime-clay stabilization system results in the production of more C-S-H gel which results in a dense paste and benefiting the system by reducing porosity, reducing permeability, increasing strength and increasing sulphate resistance. This explains the high strength values achieved with the GGBS blended mixtures (see Fig. 5.3(c); 5.5; 5.6; 5.7; 5.8 and 5.9). From the results when GGBS was incorporated into the system, a lot of benefits were observed which crystallized as enhanced strength and improved durability. From Figures 5.28 – 5.32 the presence of GGBS brought about high reduction in the permeability of samples. With the increased production of cementing gels upon the addition of GGBS, more of the pore spaces are believed to have been filled, which reduced the intrinsic permeability characteristics of the resultant materials.

6.1.2 Compaction, Density and Strength

As reported by Bahar et al (2004) the main objective of soil compaction is to increase the soil density, decrease the void ratio, reduce the soil porosity, water permeability and improve water resistance, hence, enhance its durability. However, Prusinski and Bhattacharja (1999) summarises that a given compactive effort at optimum moisture content will allow soil-stabilizer particles to achieve their closest packing and maximum density. When optimum moisture and maximum density are achieved, the soil is generally at or near its highest strength for that compactive effort. From Figure 5.2 it is clear that the density of a soil initially increases with the addition of moisture. This happens until an optimum is attained which is known as the Maximum Dry Density (MDD) and this occurs at a moisture content level that is known as the Optimum Moisture Content (OMC). At this threshold, maximum strength is achieved. This follows from the reasoning that on

increasing the moisture, the inter-particle friction is reduced and particles packing becomes optimized (Kinuthia et al (1999).

Similarly, the addition of moisture also reduces the air voids with an effect of improving the sample density. It is essential that treated soil is compacted with no more than 5% air voids as this increases strength and reduces the propensity for expansion (Greaves, 1996). These processes are related in a way that when effort is applied (compaction) it increases the packing density which in turn improves the overall bearing strength of the system. For a chemically stabilized system, the process of strength improvement with addition of stabilizer is derived from the formation of the cementing matrix structure offered by the crystallization of the C-S-H and C-A-H gels.

When lime is used as a stabilizer material, it brings along the exclusive benefit of pre-mixing otherwise known as mellowing. This allows the lime to be pre-mixed with the clay soil and allowed for up to 72 hours for the lime to react with the clay minerals. This improves the plasticity, workability and trafficability of wet plastic soils and when finally compacted, the tendency for expansion that is inherent in clay is reduced. According to Bahar et al (2004) the densification of the soil mass also makes particle reorientation and formation of cracks more difficult. This ultimately results in the durability of the compacted geo-material. On increasing the density of the test samples, better strength results were achieved as demonstrated by Figures 5.8 and 5.9 where an increase of about 13% in the sample compaction density resulted in over 53% corresponding increase in the unconfined compressive strength. Sivakumar et al (2006) reported that inter-aggregate pore spaces reduce with increasing compaction effort. However, the size of aggregates and the compaction pressure both determine the actual pore size distribution, and this has an effect on the bearing-capacity of stabilised soil components.

6.1.3 Strength Development and Curing Age

For stabilized systems, the strength development is progressive as shown by the results of the current research work. This progressive increase in strength development is related to the curing characteristics and the available time for curing. At extended curing times the

processes that increase strength gain are established and the samples become stronger and likewise become more durable (Stavridakis, 2006). Thomas (2001) reported that the strength of a lime/GGBS stabilized system is derived from the pozzolanic and hydration products. These products which are basically C-S-H and C-A-H gels, upon crystallisation, form the matrix structure on which the strength of a stabilized system is founded (Tasong et al, 1999). Clay minerals are pozzolanic in nature and when in contact with lime, in the presence of moisture, the Ca^{2+} ions from the lime combine with the silicates and aluminates from the clay to form compounds with cementitious properties (Tasong et al, 1999: Greaves, 1996; Bell, 1996). The sustainance of this reaction has been appraised by different researchers to be dependent on the availability of lime which provides the pH level of about 12.4 that is a pre-requisite for silicate and aluminate dissolution.

The available lime in a lime/GGBS stabilized system is governed by the lime demand for pozzolanic and hydration reactions upon the activation of GGBS. Hence, the addition of GGBS to a lime stabilized system as has been reported by Tasong et al (1999) and Wild et al (1998) reinforces the production of more cementitious products, enabling the enhancement of desirable engineering properties. From Figures 5.3 and 5.5, it is clear that reducing the percentage of lime addition by replacement with slag progressively improved the gain in strength until a limiting value was reached. This agrees with the postulation of Wild et al (1998), that strength will increase with increasing slag level as long as sufficient lime is present to enable alkali-slag activation. The activation of GGBS requires high temperature which is provided by the lime component upon the slaking of quicklime. According to Wild et al (1998) the increase in strength as lime is substituted with slag is predominantly due to the increased formation of the cementitious silicate, aluminate and alumina-silicates. This is reported to lead to reduced porosity of the hydrated system, as the gel develops within the pore spaces and closes off the capillary pores (Wild et al, 1998).

The compaction moisture content at which a sample is stabilised also has a big influence on the rate of strength development (refer to Figures 5.5, 5.6, and 5.8), with a moisture level addition of 28% appearing to be an optimum value where strength was optimal. Upon increasing the compaction moisture content, there was a corresponding increase in strength gain. The slaking of lime is an exothermic reaction requiring high moisture, and hydration

reaction also requires high moisture. Hence, in the Clay/lime/GGBS system there is a competition for available moisture to carry out the lime slaking process necessary for GGBS activation and the hydration reaction of lime-activated GGBS. This practically means that at low moisture contents the reaction rates were reduced while at increased moisture contents the rates of reactions were increased. This increase in the rate of reaction facilitated by the production of more hydration reaction products further enhanced the overall gain in strength.

6.1.4 Compaction Moisture Content

Bell (1996) reported that the amount of water available for hydration reaction to form cementitious bonds influences the strength which can be attained. The author also postulated that a low mix water content means that not only is insufficient water available for efficient compaction but also that water is rapidly used in the hydration process and so the maximum gain in strength is developed after a short period. In order to develop maximum strength it is necessary to use a mix water content slightly in excess of the optimum moisture content (Sivakumar et al, 2006). This allows the achievement of a satisfactory dry density with sufficient moisture available to ensure full hydration. Bell (1996) reported that if the mix water content is well in excess of the Optimum Moisture Content, then a softer soil mixture is produced but this will continue to increase in strength with increasing length of curing period. However, Osinubi and Eberemu (2006) upon the investigation of the hydraulic conductivity of compacted lateritic soil treated with blast furnace slag, concluded that increasing the moulding water content of test samples resulted in deflocculating the samples by reducing pore spaces. From the current work the moulding moisture content had great influence on several properties of the Lime-GGBS-Clay system. Increasing compaction moisture content improved strength development (see Fig. 5.5-5.9), and resulted in lower permeability values (see Figures 5.28-5.32).

On the linear expansion properties, it was found that at lower compaction moisture contents expansion were generally higher and reduced as the moulding water contents decreased (see Figures 5.46 - 5.49). For the measurement of soaked strength, higher

strength values were achieved from samples produced at higher moisture contents than those at lower moisture contacts. There are two possible causes to this effect: (1) due to the water demand of stabilized materials the abundant moisture facilitated hydration reaction between the lime activated GGBS and clay minerals, resulting to the production of more C-A-S-H gel with a pore-blocking effect, and/or (2) higher moisture content reduced particle-particle friction thereby facilitating micro-structural arrangement of particles.

6.2 Durability Parameters

Engineering projects are capital intensive, hence ensuring the durability of constructed projects does not only result in value engineering through cost savings but also ensures a safer environment. The stabilization of sulphate-bearing clay soils has always been met with the apprehension of excessive heaving resulting from the formation of the duo deleterious substances of ettringite and thaumasite. Wild et al (1999) reported very large amounts of heave, in excess of 100%, leading to pavement failure in the Stewart Avenue lime treated sub-base in Las Vegas, NV, USA and another of about 60%, on the Banbury IV contract, part of the M40 Oxford to Birmingham Motorway. This issue of expansion of lime-stabilised sulphate-bearing clay soils has been exhaustively investigated by different researchers. It has been established that blending GGBS with lime can reduce this expansive effect and bring about excellent material engineering properties through the formation of C-S-H and C-A-S-H cementing gels. However, for a scenario where flood-prone sulphate-bearing clay is stabilized with lime and lime/GGBS, very little or no literature exists. This is part of the work that the current project had intended to address; in the next section, efforts aimed at addressing the durability and practical ways of mitigating the dilapidating effects of flooding on stabilized soil materials are reported. This was achieved through the investigation of different transport mechanisms by which fluid movement in different geo-materials are conducted. It is possible to proffer a lasting solution to practices that compromise durability in stabilized systems.

6.2.1 Water Absorption

The water absorption of Lime-GGBS-Clay stabilised systems were studied in the current work. This involved the monitoring of a set of different blended mixes at some defined Compaction Moisture Contents. Different soaking regimes were followed to investigate the rate of water intake and the intake capacity of different blended stabilizers upon stabilization of Lower Oxford Clay with lime/lime-GGBS. It is worthy of note that the water absorption characteristics of a soil material are related to the pore space and void characteristics of the material, the material's affinity for water, humidity of the surrounding environment/wind characteristics and the level of compaction applied to the material. The nature of the fabric and the chemical activities taking place within the material's micro-pores are also important secondary parameters that could affect the assessment of water absorption in stabilized materials. From the results obtained from the current research, different trends were exhibited by the stabilized LOC soil material. Figures 5.13 and 5.14 show the water absorption profile of LOC stabilized with different blended mix proportions after 7-days of moist curing, during soaking in water for 10-days. High water absorption and faster rate of absorption were observed from the samples where lime was not replaced. Within the same samples, the effect of compaction moisture content was very obvious, with water absorption and the rate of absorption decreasing with increase in the compaction moisture content. This also follows the argument that as compaction moisture content increased, the friction between particles reduced to enable a more organised packing arrangement to be achieved. Secondly, increased compaction moisture content enhanced the completion of the hydration and pozzolanic reactions resulting in the production of more hydration products that increased sample densification.

From all the water absorption results, the addition of GGBS reduced the rate and amount of water absorbed. This corroborates the findings of Wild et al (1998) and Prusinski et al (1999) that the presence of lime-activated GGBS has a pore filling effect. The GGBS enhanced the production of cementitious compounds which further enhanced the densification of the Lime/GGBS stabilized clay soil. From Figure 5.13 – 5.27 the effect of lime replacement is evident. The higher the lime content the higher the moisture demand of

the system. The exothermic reaction of quicklime is known to reduce the moisture of the lime-stabilized system as much heat is generated which evaporates moisture from the soil.

Increasing the curing time also contributed positively to the water absorption characteristics of the system. It is believed that at increased curing age there was a corresponding increase in the degree to which the cementitious compounds were formed. The cementitious gels that were formed crystallised and increased the resistance to water ingress. This happened till 56 days when the reactions seem to have ended. Once the production of these cementitious gels reduced, continuous soaking in water past this age showed an increased rate and capacity of absorption. It is believed that before this peak, the integrity of the samples were maintained because even when the samples were soaked, because the hydration reaction was still ongoing, the moisture going into the system was used to satisfy the moisture needs of the system. On the contrary, because hydration has reduced to its minimum after a certain age (56 days), any moisture ingress into the system upon soaking rather presented a solubility effect thereby weakening the originally established particle bonding. This explains why initial low absorptions were possible after prolonged curing before soaking. A comparison of Figures 5.13 to 5.26 reveals that upon longer curing and subsequent soaking, the initial absorption within the first 24 hours was very low but at prolonged soaking the water absorption further increased. As identified earlier, the compaction of the stabilized material has a bearing in the water absorption characteristics. From the results it is observed that at higher compaction moisture contents the water absorption was lower. This could be as a result of the enhanced compaction due to reduced inter-particle friction. The better compaction reduced the amount, size and orientation of micro pore spaces, thereby reducing the possibility of large cracks which offer access to water accumulation in the system of the high moisture content samples.

6.2.2 Permeability

According to Barker et al (2007) stabilized materials increase their intrinsic permeability through the process of pozzolanic reactions. This is through the formation of calcium aluminate silicate hydrate (C-A-S-H) and/or calcium silicate hydrate (C-S-H) gels, and subsequently crystallising into the void spaces. Similarly, Puppala et al (2003) revealed that GGBS reduces the permeability of stabilised soil mixture as a result of improved

chemical resistance accruing from the formation of more C-S-H gel. This has the effect of causing changes in the micro-structure, pore size and pore orientation of the material. Wild et al (1996) also noted that hydration reactions have the effect of enhancing pore-blocking capability, especially where samples are cured for extended periods. This effect is useful in improving the binding and adsorption effects, and enhances resistance of GGBS concrete to sulphate attacks. The authors argued that utilising GGBS in soil stabilization will also bring similar benefits.

Bahar et al (2004) in stabilizing soil with cement found out that increasing the amount of cement from 5% to 20% decreased the coefficient of permeability from 14 x 10^{-8} to 0.27 x 10^{-8} m/s. This shows that soil stabilisation could lead to a better mechanical strength and lower permeability which represents better durability, and this corroborates the findings of the current research work. The results in Figures 5.28 to 5.32 revealed that the permeability of Lime-GGBS-Clay stabilized systems decreased with increased curing time, increased compaction moisture content and with increased lime replacement. This reduction in permeability upon extended time of curing reflects the growth of cementitious products with time (Javed and Nasim, 2005). The crystallisation of these products has been appraised in earlier sections to be responsible for sample densification and improved durability.

When the cementitious products of C-S-H and C-A-S-H crystallise, the bonding between particles is increased and internal pores are subsequently closed. When there is low interaction between pore spaces due to the crystallization of these cementitious products, the movement of fluid through the system is impeded, stalled or reduced. High permeability was recorded for the lime stabilized mixtures (see Fig. 5.28), however the addition of GGBS reduced this effect (see Figures 5.29 and 5.30). This also stems from the possibility of more cementing gels being formed upon the activation of GGBS with lime. This effect was observed by Wild et al (1996) to be responsible for the enhancement of durability of concrete prone to sulphate attack, upon stabilization with blended cement.

Increased compaction moisture content is believed to have contributed to the permeability reduction in two ways; by improving the particle packing, and facilitating hydration reaction due to sufficient moisture to support hydration reactions. Osinubi and Eberemu (2006) reported that where low permeability is require, especially in a stabilized system, a hydraulic conductivity of the order of 1×10^{-9} m/s would be appropriate even for hydraulic barrier liners. From the investigations carried out by Osinubi and Eberemu (2006), the addition of GGBS to a lime-clay stabilization system was found to offer low permeability when there was enough free lime to activate the GGBS, at increased compaction effort and increased packing density, increased moulding water content (compaction moisture content), and at extended curing ages. There is a prevailing argument that the addition of GGBS has a pore-blocking effect (Osinubi and Eberemu, 2006; James et al, 2008; Wild et al, 1999). This was observed to accrue from the formation of the various cementing products. The cementitious products are Calcium-Silicate-Hydrate (CSH), Calcium-Aluminate-Hydrate (CAH) and Ettringite according to Modmoltin and Voottipruex (2009). The formation of these products increases with increase in slag content, thus decreasing hydraulic conductivity. Osinubi and Eberemu (2006) argued that this decrease might also be due to precipitation of calcium carbonate in the voids as the ionized calcium reacts with the dissolved carbon dioxide in the water. From the findings of Murty and Krishna (2006) the average coefficient of permeability of clay treated with $CaCl_2$ was observed to be around 2.18×10^{-6} cm/s with that of untreated clay at 3.45×10^{-9} cm/s.

For the same curing ages where lime was replaced by 50% and then 75% GGBS an enhancement of about 70% and 120% was made on the permeability for the most permeable specimens. On increasing the curing period to 56 days the permeability reduced by about 100% for the 23% stabilized samples (see Fig 5.28(b)). Given a high level of lime addition, there is high possibility of carbonation especially with lower compaction moisture content (see Fig. 5.28(c)) where more un-reacted lime is presumed to still exist owing to limited moisture and available clay minerals. Upon stabilization of Lower Oxford Clay with lime the engineering and index properties of the soil material changes. This reaction process is immediate according to Bell (1996). However, on longer curing after about 72 hours the chemically active compounds of the stabilizer material combines with the clay minerals through the process of pozzolanic reaction to form compounds with

cementitious values. The gels that are formed subsequently changes with time from the initial gel state to crystalline phases that strengthen the inter-particle bonding of the stabilized material, thereby resulting in strength development.

From Fig. 5.28(a), either due to the low moisture content level of the system or lack of proper compaction due to high inter-particle friction, the samples tended to permit high water passage through the lime system than through the lime-GGBS system. The test specimens were properly controlled against moisture loses by applying several runs of cling film to regulate the rate of evaporation. However, it is difficult to assess the amount of water that was used by the system to facilitate pozzolanic reactions and the amount that effectively past through the material medium. The flow of water through the specimens increased from the start of the test at a high rate but later reduced as the system was becoming either more saturated or dense enough to resist the permeation of water. Upon increasing the compaction moisture content from 23% to 28% the flow of water through the medium was reduced by almost 50%. At this level of moisture addition, the compactions of samples were more effective. This was possible through the reduction of inter-particle friction that consequently aided the optimization of micro-pores arrangement and particle packing. Similarly, a further addition of moisture to arrive at a moisture content of 33% also showed significant reduction in the movement of fluids through the stabilized medium. At this age of curing (7-days) this effect could be attributed to the optimized compaction resulting from increased moisture and reduced particle-particle friction or the high demand for moisture to carry on with the pozzolanic processes.

The reduction in permeability at early stages of moist curing is envisaged therefore, to derive from the compaction effect on samples; the compaction is seen to be generally better at higher compaction moisture contents of 28% and 33%, due to reduced inter particle friction and improved particle parking arrangements. After several days of curing the reduction in permeability is believed to be as a result of strength enhancement through the C-S-H gel formation accruing from the replacement of lime with GGBS.

The introduction of GGBS introduced yet another form of reaction (hydration reaction). The GGBS required lime or an alkaline medium to be activated. This was provided by lime which increased the pH range of the system to 12.4 enabling the hydration reaction of activated GGBS and clay minerals to take place. However, the replacement of lime with more GGBS obviously means that the water demand of the Lime-GGBS system is reduced by half. This enhanced the compaction better than when lime was not replaced. After 7-days of moist curing the effect of GGBS in reducing the permeability may not have been as obvious, as the hydration products require time to crystallize in the pores and improve material properties. Hence, this initial material enhancement is believed to emanate from the ability of GGBS to reduce water demand through dilution of lime. The presence of GGBS has been very useful in reducing the permeation properties. This is through the formation of more cementitious products after extended curing. Secondly, the presence of GGBS enhanced the compaction characteristics, where better compaction was achieved with improved durability and strength properties. However, the most important factor in lime replacement is to ascertain the level of replacement of lime that could still provide enough free lime in the system to activate the GGBS.

6.2.3 Volume stability (Linear Expansion)

Clay minerals exhibit medium to high volume change upon wetting. This large volume change will upon wetting cause extensive damage to structures, in particular, light buildings and pavements (Rajasekaran, 2005). This deleterious effect has been evaluated to stem from the settlement characteristics of clay soil upon alternating seasons of moisture variations (Stavridakis, 2005; Hafez et al, 2008). When lime is used to stabilize sulphate bearing clay soil, the risk of heaving is increased (Puppala et al, 2003). This heaving of sulphate bearing clays upon stabilization with lime has been evaluated to emanate from the formation of deleterious minerals such as ettringite and/or thaumasite (Puppala et al, 2003; Wild et al, 1996; Oti et al, 2008; Rajasekaran, 2005). According to Oti et al (2008), ingredients in some clay soils, for example pyrites and gypsum may aid the formation of these deleterious substances. However, investigations by Wild et al (1999) revealed that GGBS hydration consumes lime thereby reducing expansion in a Lime-GGBS-Clay soil stabilized system. Wild et al (1996) and (1998) also reported that sulphate expansion

associated with ettringite formation can be prevented by replacing cement with GGBS. This is true from Figures 5.46 and 5.48; it could be observed that the replacement of lime with GGBS reduced the capacity to expand. Increasing the compaction moisture content is also effective in reducing expansion.

The curing age did not seem to have much effect on the expansion behaviour of the lime replaced samples after the initial 7 days of moist curing. It is believed that at this age the matrix structure from which strength depends is not well established. Secondly, upon the extrusion of the samples from the cylindrical mould there will be some relaxation effect from the samples after having been subjected to some static compaction. This relaxation effect could be very fast in highly elastic materials. However, in materials of low elasticity, this effect even though it may be small could drag for days. With the high sensitivity of the testing apparatus, it was possible to pick up these signals as part of the linear expansion of the test material. Hence the early expansion could be pseudo-expansion, and it is therefore reasonable to argue that without this initial rebound there would be less than 2% of expansion which meets standards for expansion of stabilized materials. The lime stabilised samples showed high expansion potential and the expansion increased with soaking age. This could be as a result of the formation of deleterious mineral ettringite resulting from the sulphate content of the soil or the oxidation of pyrite.

6.2.4 Effect on Strength and Durability upon Soaking

A durability index assessment of the various mix proportions were carried out as a measure of the resistance to environmental damaging factors. Different experimental regimes were followed, and results showed that high exposure to these environmental degrading forces has a corresponding effect on the extent of damage observed. The results in Figures 5.33 to 5.39 show an overall trend of increasing strength with curing age. This follows the argument presented by different investigators that age of curing supports the formation and crystallisation of the cementitious gels that enhance strength (Al-Rawas et al, 2005; Puppala et al, 2003; Wild et al, 1996; etc).

Even though the test samples were soaked in water after specific curing ages, the resistance to the effect of the simulated flooding still increased. The curing of samples increased their strength with age up till 56 days of curing and then soaking. The reduction in strength of the soaked samples was very low when compared with the unsoaked samples. At this stage of the strength development, soaking the samples in water meant fulfilling their demand for water to carry out the hydration reactions. This explains why within this period the loss in strength upon soaking was minimal. As has been reported by Bell (1996) and Sivakumar et al (2006), large amounts of water are required to carry on hydration reactions from which the cementitious products are formed. However, after 56 days of curing the rate of hydration reduces because of the depletion of free lime to carry out hydration reactions (see Fig. 5.35). This resulted in high loss in strength upon soaking.

Partial soaking of samples also meant reduced availability of water to the test samples. This produced high strength index values as shown in Figures 5.34 and 5.35 where the partially soaked samples also showed higher strength than the unsoaked after some days of curing (7 to 28 days) and 4 days partial soaking. Soaking is very critical for lime stabilized mixtures. Given the high water demand for lime stabilized mixtures, the tendency is that the interstices absorb much water which reduces bond strength. This is even more critical at the early ages of strength development (see Fig. 5.33). At extended soaking of up to 10 days the durability fell lower than that of 4 days of soaking. In these cases the 4%L-12%G and 8%L-8%G samples stabilized at 28% showed higher resistance to the strength reducing effects of water (see Fig. 5.37 and 5.38).

Stabilized materials are characterized by increased strength development even under soaking conditions. The addition of GGBS enhanced the durability index parameters of stabilized materials. However, the durability index reduced with increased age of curing and soaking. The unsoaked samples increased in strength progressively, within the early periods of soaking, where the effect is minimal hence a corresponding effect on the durability. On extended curing to the point where the rate of hydration is very low (after 28 days), the effect of soaking was more noticeable as it compared negatively with the high strength values of the unsoaked samples (see Fig. 5.40 to 5.43). Increasing lime

replacement improved durability as is shown in Fig. 5.40 to 5.43. Partial soaking of samples resulted in improved durability index (see Fig. 5.42 and 5.43). The effect of extended soaking on soaked strength values is also replicated in the durability index, in that higher soaking or exposure to the soaking environment represented a huge reduction in the durability measurement, with higher influence on the lime stabilized mixtures (see Fig. 5.44 and 5.45).

6.2.5 Compressibility upon loading

According to Rajasekaran and Rao (2002) the compressibility assessment of a soil is important as the load causing unacceptable settlement in soft clays may be less than the load required to cause shear failure. The soil may be strong enough to resist shear failures according to the authors, but the settlements may be quite large and often exceed the permissible limits. Rajasekaran and Rao (2002) reported the various mechanisms controlling the compressibility behaviour of clays. However, these mechanisms could be altered by the application of a lime stabilizer. The main factors influencing the volume change behaviour of soils are the bond strength, inter-particle forces, dielectric constant, mineralogical composition and chemistry of the pore water system, among other factors (Rajasekaran and Rao, 2002).

Different investigators have reported on the beneficial effect of applying lime and/or lime-GGBS to soft clays. According to Rajasekaran and Rao (2002), lime-soil reactions yield various reaction products. These products are initially in a gel state and then change slowly into crystalline phases. During the process of phase conversion, soil particles bind together and result in the growth of a cementitious matrix. These cementitious compounds have been noted to create a bridging effect and the resulting cementation bonds formed in the system resist compressibility forces more effectively. From the results presented in Table 5.4 the compressibility is observed to reduce with curing age. This is in agreement with the fact that strength development is time dependent, as the cementitious products transform from one phase to the other and thus resistance to compressibility increases. When lime is not replaced, the test samples were very vulnerable to soaking in water. This resulted in

early high compressibility with the effectiveness of the resistance to compressibility forces improving with time as cementitious compounds crystallised in the pore spaces. Rajasekaran and Rao (2002) argued that the improvement in compressibility is mainly due to the lime-induced changes occurring in the cationic system and aggregation of soil particles as brought by soil-lime reaction.

The compaction moisture content at which samples were moulded has also shown a big influence in the compressibility of the entire test samples, except those where lime was not replaced. At higher compaction moisture contents better compaction was achieved due to the improved packing of particles. This reduced inter-particle pore spaces is reported by Sivakumar et al (2006). When the void volume is low the compressibility will be low. Under the same applied loading, the level of compressibility obtained from the lime-GGBS blended mixes is minimal when compared with that obtained from the lime stabilised LOC upon 10 days complete submersion in water and subsequent loading of samples.

6.2.6 Regression Analysis

Adopting the secondary data analysis option by Surendran (2001), Multi regression analysis was carried out using 'MINITAB'. This statistical software manages raw data and develops regression equations of different degrees as well as best fits to graphs. In the regression analysis where the 'Response' (key element) was Strength, variables such as: age of curing, compaction moisture content, and the compaction density were considered. These variables were tested for correlation; it was shown that they were not correlated to each other. Hence, they were all included in the equation; however, on carrying out the regression analysis, the compaction density which was essentially constant was removed from the equation, with the following relationship established for calculating strength for the different stabiliser compositions at a density of 1.79Mg/m^3.

UCS (kN/m^2) = 493 + 15.6 Curing Age (Days) - 1.85 Compaction m/c (%)

And at an increased density of 2.04Mg/m^3 the following equation applies:

UCS (kN/m^2) = 3218 + 30.3 Curing Age (Days) - 66.4 Compaction m/c (%)

These equations satisfy the requirement of P-value less than 0.05 which shows that the parameters considered in the equation are significant, and they also have R^2 values of 7.4% and 90.8% respectively, for details of the multi regression analysis see appendix F.

Similarly, the relationship between the unsoaked; partially soaked; and completely soaked strengths against the different variables of curing age, compaction moisture content, and compaction density were also established at a density of $2.04Mg/m^3$. In this case an R^2 value of 55.5%, 26.3%, and 20.0% respectively were obtained, which is very significant and the P-values were also less than 0.05. For the strength against compaction as shown in the graph in appendix F, a negative correlation was observed, while it was positive for strength against curing age. The following regression equations were however established for the different parameters.

UCS Unsoaked (kN/m^2) = 3717 + 22.1 Curing Age (Days) - 88.6 Compaction m/c (%)

Partially soaked (kN/m^2) = 2909 + 16.6 Curing Age (Days) - 66.7 Compaction m/c (%)

Completely soaked (kN/m^2) = 2131 - 41.7 Compaction m/c (%) + 12.1 Curing Age (Days)

Based on the regression equations, it was possible to simulate the strengths that are likely by applying different compaction moisture contents, at different curing ages, and varied soaking influences. Figure 6.1 shows the simulated strength using the regression equation

for both low and high compaction densities while Figure 6.2 shows the strength after different soaking regimes. The trend of increasing strength with curing age was observed in both cases of low and high compaction densities as well as with the varied soaking systems. However, upon increasing the density from 1.79Mg/m^3 to 2.04Mg/m^3 as shown in Fig. 6.1 the strength was significantly enhanced. This further shows how critical curing age is in the development of high strength within stabilised systems. This is equally evident from Figure 6.2 where the strength achieved by the unsoaked and partially soaked systems equalled or surpassed that achieved by the completely soaked system. Strength of about 3000 kN/m^2 was achieved and surpassed by the unsoaked and partially soaked within the first 20 days of curing; however, this could only be achieved in the completely soaked system after about 70 days of curing.

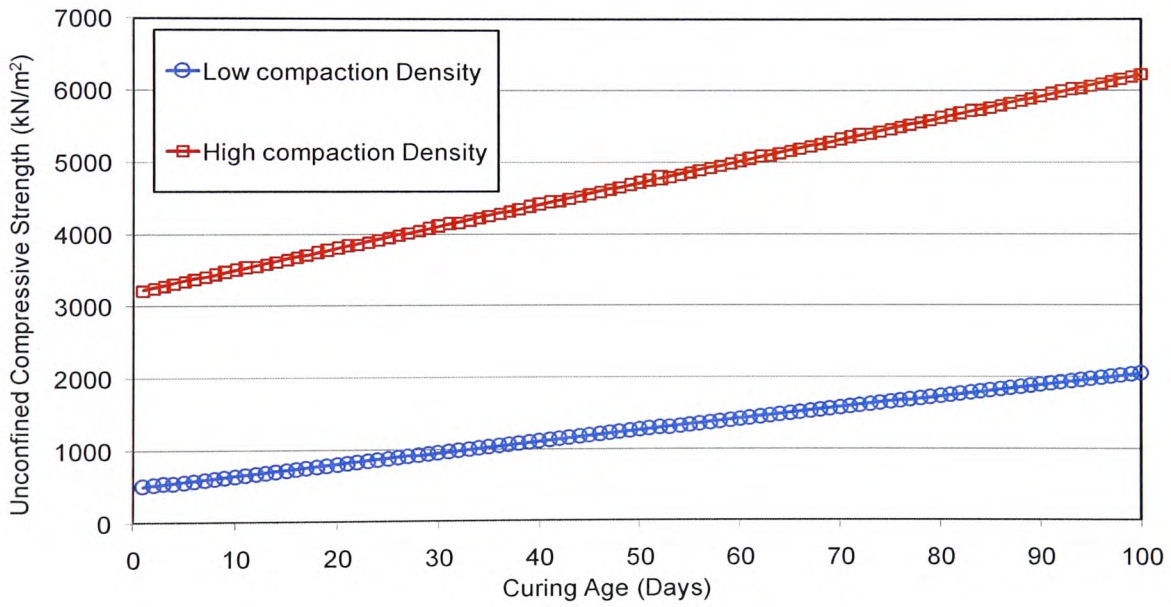


Fig. 6.1 Strength simulation of varying compaction density based on regression equation

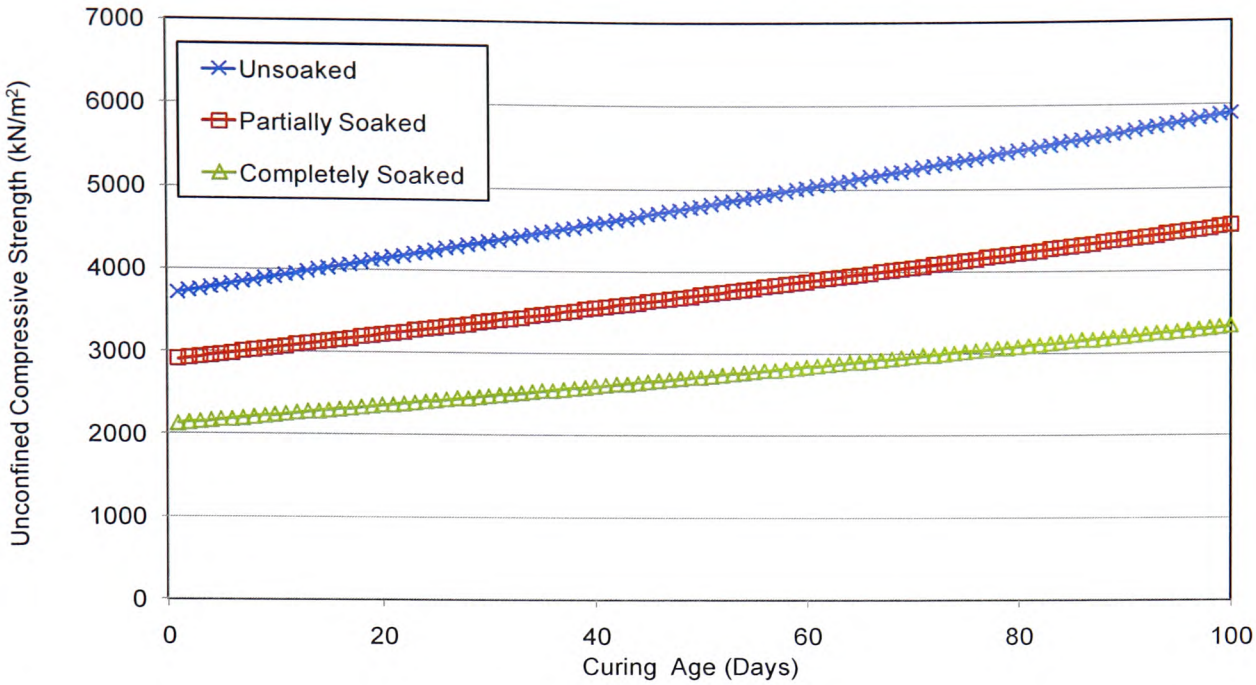


Fig. 6.2 Strength simulation of different soaking regimes based on regression equation

6.2.7 Cost-Benefit-Risk profile

To carry forward the sustainability argument presented in this thesis, a cost-benefit-risk analysis of the novel material formulation was assessed against classical materials of lime and Portland cement as shown in Table 6.1. Lime replaced with 75% by-product materials showed the lowest cost when compared with Ordinary Portland cement and Lime. Similarly, there are other benefits and risk which are not easily quantifiable and hence would be tricky to be assigned any monetary value. These include; the amount of landfill space created and the hazards associated with them, the cost of carbon dioxide emission per ton, safety/risk of usage, strength achievable, and durability criteria. Using a high, medium, and low effect in assessing these parameters, the 75% lime replaced blend was still adjudged to possess the highest benefit to users. The production of lime and Portland cement are associated with high taxes and fees which vary from locality to locality and significantly influences the overall cost of the finished products (lime and Portland cement). However, GGBS as a by-product material is produced at no associated tax or fees. The use of cement is associated with high risk of health and safety, and because of the vigorous reaction of quicklime with water, quicklime causes severe irritation when inhaled

or placed in contact with moist skin or eyes. Inhalation may cause coughing, sneezing, laboured breathing e.t.c. There has been no known risk of health and safety relating to GGBS to date, making the use of GGBS a low risk option compared to lime and cement, hence, the replacement of lime with GGBS will greatly reduce the health and safety risk associated with the use of lime un-replaced.

Table 6.1 Cost-benefit-analysis of Utilising activated-GGBS in the stabilisation of flood prone soil as against the use of Cement and Lime.

	natural materials required/ton (US Dollars)	Energy requirement/ton (US Dollars)		Total cost per ton (US Dollars)
		production	Grinding	
Cement	31.87	10	3.50	45.37
Lime	31.87	6	3.5	41.37
Lime/GGBS 50 : 50	15.94	3	3.5	22.44
Lime/GGBS 25 : 75	10.62	2	3.5	16.12

*These are prices as at September of 2011.

6.2.7.1 Cost Annualisation

Engineering economic analysis offers different tools for the evaluation of investment alternatives. To adjudge a system as sustainable and guide decision makers in executing sound judgements that otherwise merit the investment on certain projects, it is imperative that the Total Capital Cost on investment, services, or different projects implementation be evaluated for better return on investment. Considering the 'Total Annual Cost', this is the equivalent annual cost of owning and operating an asset or investment (project) over its entire lifespan. For a better understanding of the concept of cost annualisation, the following need to be defined:

Annualised Capital Cost – this is the annual 'mortgage' which over the lifetime of an investment, is able to repay the total cost of investment. Annualised capital cost is therefore, the capital cost multiplied by annualisation factor and the annualisation factor depends on interest rate and the life span of a given project or investment alternative.

Total Annualised Cost – this is the addition of the annualised operational cost to the Annualised Capital Cost. The Total Annualised Cost which could also be referred to as Equivalent Annual Cost (EAC) is expressed as:

$$EAC = \frac{\text{Asset Price x Discount Rate}}{\text{1-(1+ Discount Rate)}^{\text{-number of period}}}$$

This could otherwise be re-written as:

$$EAC = NPV/A(t,r)$$

Where NPV or NPW is the net present value or net present worth, A is the present value of Annuity Factor, t is the time of lifespan in years, and r is the rate of loan or capital cost.

A cost-benefit analysis of the traditional stabilisers and those of the novel material formulation has been carried out. However, to make the findings from the current research readily usable, it is necessary to perform a cost annualisation of the envisage capital cost. The material cost of the system with 75% replacement of traditional stabilisers with GGBS was considered. With a capital cost of about 16.12 US dollars per/ton of material used in a project with a design service life of 20 years which is typical of a road project, with low maintenance cost at about 4.30 US dollars per/ton of material utilised. Adopting an interest rate of 5% over a 20 years period will give 0.080 Annuity Factor from table (see Appendix F1). Therefore, to finance an asset or project costing 20.42 US Dollars per/ton for 20 years, the annual investment per/ton will be 1.634 US Dollars.

6.2.8 Practical Implications

Flood-prone low-bearing capacity soils are well known for their moisture sensitivity. With the volume of soil increasing upon wetting and shrinking when dried, the effects of these

changes on road structural layers are usually enormous as the strength is compromised and durability reduced. The isolation of large areas due to inaccessibility has a huge safety, environmental and economic burden placed on scarce resources. The current research has a practical applicability in this respect as the high moisture sensitivity of these materials is reduced, leading to improved strength and durability enhancement, and prolonging the duration under which a stabilised material can remain submerged in water without complete disintegration. The work has also wider implications, as the very strong material has application in the building industry where low-cost building components (such as bricks, blocks and mortar) of high resistance to climatic changes (the effect of erosion due to flash flooding or splashing water during heavy rains) can be formulated. This reduces cost of maintenance with reduced negative environmental impact. The use and reuse of natural, industrial and other waste streams, such as agricultural waste, not only saves the scarce natural resources but also offer a reliable and sustainable step towards a cleaner and safer environment.

Chapter Seven: Conclusions and Recommendations for further work

7.1 Conclusion

Water is a big problem to soil and constructed road structural layers. For self draining soils such as sand, the effect of high moisture could be relatively low when compared to clay soil where drainage is very slow with the potential of building pore water pressures. The moisture sensitivity of clay soils could lead to low-bearing resistance with associated problems such as loss in bearing-strength, increased shear failure and volume instability, leading to low durability among other problems. From this investigation, it was found that compressive strength, water draining characteristics and durability criteria for assessing weak geo-materials were improved by the application of a classical stabilizer (lime), significantly replaced with an industrial by-product material (GGBS), to up to 75% replacement. The following conclusions could be drawn from the evidence, facts and figures obtained from the current research:

1.) It was established that, there is a relationship between the strength and the durability aspects of stabilized soil systems. This follows that high strength almost always translates to better durability. The growth of reaction products were encouraged by extending the curing time and this improved the bearing strength of stabilized materials. GGBS enhanced the strength of a Lime-Clay system through the production of more cementitious gels to fill up the pore spaces. When the cementitious gels pass through a phase changing process they become relatively more crystalline and increase the bonding strength between particles to enhance overall bearing strength and durability. The strength of the Lime-GGBS-Clay system increased when there was enough free lime to activate the GGBS. However, when a stabilizer dosage of about 16% was used, the strength of the system increased upon gradual replacement of lime with GGBS to attain a Lime-GGBS blended ratio of 1:3. The trend from the regression analysis shows that strength depends largely on curing age and the moisture content at which specimens were compacted.

2.) High moisture content improved the compaction of a Lime-GGBS-Clay stabilized system. This resulted from the reduced inter-particle friction which enabled a more

optimized particle packing arrangement. Increasing the compaction moisture contents of the Lime-GGBS-LOC stabilized system improved strength, reduced water absorption, reduced permeability. The Lime-GGBS-Clay stabilized system offered the best performance at a compaction moisture content of about 28%. At this moisture content, the GGBS was effectively activated to provide enhanced hydration reactions. When lime was not replaced by GGBS in the Lime-Clay stabilized system, the resultant stabilised material was very vulnerable to water ingress, with high water absorption capacity, and lower soaked strengths. However, upon the replacement of lime with GGBS the rate of water absorption was kept low and better soaked strengths were consequently achieved.

3.) At prolonged curing times the initial water absorption of the lime-GGBS-Clay stabilized system was very low. However, when the soaking period increased the resistance to water ingress was reduced and water absorption increased at a faster rate. This is believed to have resulted from the reduced rate of hydration whereby any water going into the system was no longer utilized for hydration but played a dissolution role to weaken bonds and compromise strength and durability. However, The Lime-GGBS-Clay stabilized system showed strength development even when soaked in water, and the resistance to water penetration upon soaking increased with curing age up to an optimum curing value of 56 days. After this curing period, the rate of strength development reduced as well as the reduction of the ratio of soaked strength to unsoaked strength. Further, when stabilized samples were partially soaked in water, the damaging effect was reduced when compared with complete immersion.

4.) The permeability of the Lime-GGBS-Clay stabilized systems decreased with i) extended curing period, ii) increased compaction moisture content and iii) with increased replacement of lime, till the limiting ratio of lime to GGBS reached 1:3. High volume instability is likely with lime stabilization. However, upon the replacement of the lime with GGBS the free lime that would have otherwise been used in the development of the deleterious compounds of ettringite and thamausite was used in the process of activating the GGBS. This produced more C-S-H and C-A-S-H gels that are known to be required for the maintenance of the integrity of stabilized geo-materials.

5.) The LOC-Lime-GGBS system produced optimal strength results compared with LOC-Lime or LOC-PC-GGBS systems. The strength was optimised when the test specimens were stabilized at a moisture content of 28%. The GGBS-based system also showed superior linear expansion properties compared to the LOC-Lime system, with the systems stabilised using 4%Lime-12%GGBS and 8%Lime-8%GGBS blended stabilisers showing best performance. In these systems, a high pH resulting with the dissolution of silicates and aluminates by lime, results in increasing the production of the C-S-H and C-A-S-H gels, which further supported strength and durability enhancement through the reduction of pore spaces, volume stability, increased density and improved intrinsic permeability.

6.) The most successful formulation investigated was 4%Lime-12%GGBS which suggested a potentially significant reduction in cost and the current over-reliance on the traditional binders – lime and/or Portland cement to stabilise soils. The combined qualities of high strength, minimal expansion and the economy associated with the incorporation of waste and/or by-product materials suggests commercial viability and a prolonged mitigation of the currently dwindling natural raw material resources. Combined technological, environmental and economic advantages in the formulations investigated strengthen sustainability, especially in terms of applications in road construction in low-lying areas where good strength development coupled with enhanced water resistance upon flooding is essential for the maintenance of a robust road network. Therefore I conclude that this research offers a low-cost option of stabilisation relative to traditional systems of soil stabilisation.

7.2 Recommendations for further work

Different areas have been identified for immediate and long term investigations, as time will not permit to completely exhaust all the likely lines of thought that attract attention for further research. Hence, it is therefore recommended that the following concerns raised by the current research work may be considered. This will give an enhanced robustness to this recent innovation and increase the confidence of practitioners and the general civil engineering community to readily adopt the proposed sustainable multi-binder options in construction works, especially for roads in flood plains and other flood-prone situations.

1.) It would be pragmatic to establish the implications of the interaction of GGBS-Lime stabilised soil with contaminated flood water, since the flood simulation in the laboratory was with normal tap water which is very unlikely for a flooding scenario.
2.) Further research should be carried out to investigate the likely outcome of repeated flooding and drying on GGBS-Lime stabilised materials as the number of flooding occasions within the service life of these novel materials after construction is likely to be repetitive.
3.) In this research work a typical flash flooding scenario was assumed which could last between one and ten days. An investigation is therefore necessary to ascertain what happens in a wide flood condition which will certainly last beyond the target period of ten days.
4.) Establishing an empirical relationship between the CBR for roads and the optimum UCS requirement will be useful as a guide to practitioners, for predicting CBR values where UCS was adopted as the strength assessment approach.
5.) In carrying out the regression analysis, the limited size of data could be associated with uncertainties, it is therefore necessary to employ wide range of data to reduce possible errors due to sample size.

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Appendix A

Calculation Sheet for 350g weight of sample

The masses of 350 and 400g were determined by first, carrying out a Proctor Compaction Test to assess the density and furthermore, calculating the volume of sample thus:

Diameter of cylinder = 50mm

Height = 100mm

Therefore,

Area = $\pi D^2 \div 4 = 3.142 \times (50^2) \div 4 = 3.142 \times 2500 \div 4 = 1963.75 mm^2$

Hence, Volume = $1963.75 \times 100 = 196375 mm^3$

$196375 \div 1000 = 196.3 m^3$

And density = $1.79 Mg/m^3$

Density = Mass/Volume, Mass = Density × Volume

Mass = $1.79 \times 196.3 = 351.377g$

This was rounded up to 350g.

Similarly for a density of $2.0479 Mg/m^3$ with the other parameters remaining unchanged a mass of 400g was arrived at.

23% moisture content

Soil + Stabiliser + Water = 350g

Ws + Wst + Ww = 350g

Ws + 0.16Ws + 0.23(Wst + Ws) = 350g

Ws + 0.16Ws + 0.23(0.16Ws) + 0.23Ws = 350g

Ws + 0.16Ws + 0.0368Ws + 0.23Ws = 350g

1.4268Ws = 350g

Ws = 350/1.4268 = 245.30

Ws = 245.30g

Wst = 0.16 × 245.30 = 39.25g

Ww = 0.23(Wst + Ws) = 0.23(245.30 + 39.25)

Ww = 65.45g

28% moisture content

Soil + Stabiliser + Water = 350g

Ws + Wst + Ww = 350g

Ws + 0.16Ws + 0.28(Wst + Ws) = 350g

Ws + 0.16Ws + 0.28(0.16Ws) + 0.28Ws = 350g

Ws + 0.16Ws + 0.0448Ws + 0.28Ws = 350g

1.4848Ws = 350g

Ws = 350/1.4848 = 235.72g

Ws = 235.72g

Wst = 0.16 × 235.72 = 37.72g

Ww = 0.28(Wst + Ws) = 0.28(235.72 + 37.72)

Ww = 76.56g

33% moisture content

Soil + Stabiliser + Water = 350g

Ws + Wst + Ww = 350g

Ws + 0.16Ws + 0.33(Wst + Ws) = 350g

Ws + 0.16Ws + 0.33(0.16Ws) + 0.33Ws = 350g

Ws + 0.16Ws + 0.0528Ws + 0.33Ws = 350g

1.5428Ws = 350g

Ws = 350/1.5428 = 226.86g

Ws = 226.86g

Wst = 0.16 × 226.86g = 36.30g

Ww = 0.33(Wst + Ws) = 0.33(226.86 + 36.30)

Ww = 86.84g

38% moisture content

Soil + Stabiliser + Water = 350g

Ws + Wst + Ww = 350g

Ws + 0.16Ws + 0.38(Wst + Ws) = 350g

Ws + 0.16Ws + 0.38(0.16Ws) + 0.38Ws = 350g

Ws + 0.16Ws + 0.0608Ws + 0.38Ws = 350g

1.6008Ws = 350g

Ws = 350/1.6008 = 218.64g

Ws = 218.64g

Wst = 0.16 × 218.64 = 34.98g

Ww = 0.38(Wst + Ws) = 0.38(218.64 + 34.98)

Ww = 96.38g

Calculation Sheet for 400g weight of sample

23% moisture content

Soil + Stabiliser + Water = 400g

Ws + Wst + Ww = 400g

Ws + 0.16Ws + 0.23(Wst + Ws) = 400g

Ws + 0.16Ws + 0.23(0.16Ws) + 0.23Ws = 400g

Ws + 0.16Ws + 0.0368Ws + 0.23Ws = 400g

1.4268Ws = 400g

Ws = 400/1.4268 = 280.35

Ws = 280.35g

Wst = 0.16 × 280.35 = 44.86g

Ww = 0.23(Wst + Ws) = 0.23(280.35 + 44.86)

Ww = 74.80g

28% moisture content

Soil + Stabiliser + Water = 400g

Ws + Wst + Ww = 400g

Ws + 0.16Ws + 0.28(Wst + Ws) = 400g

Ws + 0.16Ws + 0.28(0.16Ws) + 0.28Ws = 400g

Ws + 0.16Ws + 0.0448Ws + 0.28Ws = 400g

1.4848Ws = 400g

Ws = 400/1.4848 = 269.40g

Ws = 269.40g

Wst = 0.16 × 269.40 = 43.10g

Ww = 0.28(Wst + Ws) = 0.28(269.40 + 43.10)

Ww = 87.50g

33% moisture content

Soil + Stabiliser + Water = 400g

Ws + Wst + Ww = 400g

Ws + 0.16Ws + 0.33(Wst + Ws) = 400g

Ws + 0.16Ws + 0.33(0.16Ws) + 0.33Ws = 400g

Ws + 0.16Ws + 0.0528Ws + 0.33Ws = 400g

1.5428Ws = 400g

Ws = 400/1.5428 = 259.27g

Ws = 259.27g

Wst = 0.16 × 259.27 = 41.48g

Ww = 0.33(Wst + Ws) = 0.33(259.27 + 41.48)

Ww = 99.25g

38% moisture content

Soil + Stabiliser + Water = 400g

Ws + Wst + Ww = 400g

Ws + 0.16Ws + 0.38(Wst + Ws) = 400g

Ws + 0.16Ws + 0.38(0.16Ws) + 0.38Ws = 400g

Ws + 0.16Ws + 0.0608Ws + 0.38Ws = 400g

1.6008Ws = 400g

Ws = 400/1.6008 = 249.88g

Ws = 249.88g

Wst = 0.16 × 249.88 = 39.98g

Ww = 0.38(Wst + Ws) = 0.38(249.88 + 39.98)

Ww = 110.15g

Appendix B

B.1a. Unconfined Compressive Strength (UCS) test results for LOC stabilized at 23% moisture content with varying dosages of Lime, Lime/GGBS, PC and PC/GGBS blended mixes.

Lime (%)	7Days	14Days	21Days	28Days
0	217.3	200.32	199.3	198.28
4	520.84	591.46	660.9	730.33
8	499.77	593.84	658.35	722.86
12	519.14	559.89	569.74	579.58
16	371.79	417.62	425.94	434.26

Lime-GGBS (%)	7Days	14Days	21Days	28Days
16Lime-0GGBS	371.79	417.62	425.94	434.26
12Lime-4GGBS	566.68	800.27	873.78	947.29
8Lime-8GGBS	787.71	908.25	1005.69	1103.13
4Lime-12GGBS	807.41	981.92	1097.19	1212.46
0Lime-16GGBS	232.92	295.73	389.61	483.49

PC (%)	7Days	14Days	21Days	28Days
0	217.3	200.32	199.28	198.28
4	265.51	288.26	348.53	408.79
8	507.6	553.44	595.71	637.98
12	590.1	687.21	758.06	828.91
16	706.9	806.04	913.74	1021.43

Stabilizer	7Days	14Days	21Days	28Days
4%Lime	520.84	591.46	660.9	730.33
4%Lime-12%ggbs	807.41	981.92	1097.19	1212.46
16%PC	706.9	806.04	913.74	1021.43
4%PC-12%ggbs	784.32	958.84	1064.46	1170.07

PC-GGBS (%)	7Days	14Days	21Days	28Days
16%PC-0%GGBS	706.9	806.04	913.74	1021.43
12%PC-4%GGBS	709.62	831.17	872.9	914.63
8%PC-8%GGBS	729.99	819.05	888.44	957.82
4%PC-12%GGBS	784.32	958.84	1064.46	1170.07
0%PC-16%GGBS	232.92	295.73	389.61	483.49

B.1b. Unconfined Compressive Strength (UCS) test results for LOC stabilized at various moisture contents with varying blend compositions.

23% Moisture cont.

Stabilizer	7Days	14Days	21Days	28Days
16Lime-0GGBS	371.79	417.62	425.94	434.26
12Lime-4GGBS	566.68	800.27	873.78	947.29
8Lime-8GGBS	787.71	908.25	1005.69	1103.13
4Lime-12GGBS	807.41	981.92	1097.19	1212.46
0Lime-16GGBS	132.92	195.73	239.61	283.49

28% moisture cont.

Stabilizer	7Days	14Days	21Days	28Days
16%Lime-0%GGBS	368.71	401.36	446.77	492.18
12%Lime-4%GGBS	683.67	963.6	1043.2	1122.79
8%Lime-8%GGBS	834.69	1158.17	1265.31	1372.45
4%Lime-12%GGBS	954.08	1247.96	1361.23	1474.49
0%Lime-16%GGBS	52.04	65.82	80.11	94.39

33% moisture cont.

Stabilizer	7Days	14Days	21Days	28Days
16%Lime-0%GGBS	270.75	329.93	382.32	434.70
12%Lime-4%GGBS	565.99	796.26	935.715	1075.17
8%Lime-8%GGBS	673.81	1043.2	1261.74	1480.27
4%Lime-12%GGBS	738.1	1079.59	1357.99	1636.4
0%Lime-16%GGBS	43.37	43.37	60.72	78.06

38% moisture cont.

Stabilizer	7Days	14Days	21Days	28Days
16%Lime-0%GGBS	216.33	280.61	305.44	330.27
12%Lime-4%GGBS	556.46	796.6	852.39	908.17
8%Lime-8%GGBS	764.29	1074.83	1085.89	1096.94
4%Lime-12%GGBS	898.24	1279.93	1355.44	1430.95
0%Lime-16%GGBS	26.22	29.08	38.52	47.96

B.1c Strength development of Lower Oxford Clay stabilized using various Lime-GGBS blended mixes at varying compaction moisture contents.

23% Moisture cont.

Stabilizer	7Days	14Days	28Days
16%Lime-0%GGBS - 23%mc	371.79	417.62	434.26
12%Lime-4%GGBS - 23%mc	566.68	800.27	947.29
8%Lime-8%GGBS - 23%mc	787.71	908.25	1103.13
4%Lime-12%GGBS - 23%mc	807.41	981.92	1212.46
0%Lime-16%GGBS - 23%mc	132.92	195.73	283.49

28% moisture cont.

Stabilizer	7Days	14Days	28Days
16%Lime-0%GGBS - 28%mc	368.71	401.36	492.18
12%Lime-4%GGBS - 28%mc	683.67	963.6	1122.79
8%Lime-8%GGBS - 28%mc	834.69	1158.17	1372.45
4%Lime-12%GGBS - 28%mc	954.08	1247.96	1474.49
0%Lime-16%GGBS - 28%mc	52.04	65.82	94.39

33% moisture cont.

Stabilizer	7Days	14Days	28Days
16%Lime-0%GGBS - 33%mc	270.75	329.93	434.70
12%Lime-4%GGBS - 33%mc	565.99	796.26	1075.17
8%Lime-8%GGBS - 33%mc	673.81	1043.2	1480.27
4%Lime-12%GGBS - 33%mc	738.1	1079.59	1636.4
0%Lime-16%GGBS - 33%mc	43.37	43.37	78.06

38% moisture cont.

Stabilizer	7Days	14Days	28Days
16%Lime-0%GGBS - 38%mc	216.33	280.61	330.27
12%Lime-4%GGBS - 38%mc	556.46	796.6	908.17
8%Lime-8%GGBS - 38%mc	764.29	1074.83	1096.94
4%Lime-12%GGBS - 38%mc	898.24	1279.93	1430.95
0%Lime-16%GGBS - 38%mc	26.22	29.08	47.96

12%Lime-4%GGBS

Stabiliser blend	7Days	14Days	28Days
12%Lime-4%GGBS - 23%mc	566.68	800.27	947.29
12%Lime-4%GGBS - 28%mc	683.67	963.60	1122.79
12%Lime-4%GGBS - 33%mc	565.99	796.26	1075.17
12%Lime-4%GGBS - 38%mc	556.46	796.60	908.17

8%Lime-8%GGBS

Stabiliser blend	7Days	14Days	28Days
8%Lime-8%GGBS - 23%mc	787.71	908.25	1103.13
8%Lime-8%GGBS - 28%mc	1372.45	1158.17	1372.45
8%Lime-8%GGBS - 33%mc	673.81	1043.20	1480.27
8%Lime-8%GGBS - 38%mc	764.29	1074.83	1096.94

4%Lime-12%GGBS

Stabiliser blend	7Days	14Days	28Days
4%Lime-12%GGBS - 23%mc	807.41	981.92	1212.46
4%Lime-12%GGBS - 28%mc	954.08	1247.96	1474.49
4%Lime-12%GGBS - 33%mc	738.10	1079.59	1636.40
4%Lime-12%GGBS - 38%mc	898.24	1279.93	1430.95

B.1e. Lower Oxford Clay stabilized with varying proportions of Lime/GGBS blended mixes at different moisture contents and increased compaction density.

28% moisture content

Stabilizer Composition	7Days	14Days	21Days	28Days
4L - 12G	1599.32	1904.08	2134.87	2365.65
8L - 8G	1566.67	1874.15	1969.22	2064.28

33% moisture content

Stabilizer Composition	7Days	14Days	21Days	28Days
4L - 12G	1161.22	1434.35	1639.97	1845.58
8L - 8G	1078.23	1310.54	1531.29	1752.04

38% moisture content

Stabilizer Composition	7Days	14Days	21Days	28Days
4L - 12G	950.00	1332.99	1517.52	1702.04
8L - 8G	863.95	1155.61	1267.77	1379.93

Comparing 4L - 12G at varying mc

Stabilizer Composition	7Days	14Days	21Days	28Days
4L - 12G - 28%mc	1599.32	1904.08	2134.87	2365.65
4L - 12G - 33%mc	1161.22	1434.35	1639.97	1845.58
4L - 12G - 38%mc	950.00	1332.99	1517.52	1702.04

Comparing 8L - 8G at varying mc

Stabilizer Composition	7Days	14Days	21Days	28Days
8L - 8G - 28%mc	1566.67	1874.15	1969.22	2064.28
8L - 8G - 33%mc	1078.23	1310.54	1531.29	1752.04
8L - 8G - 38%mc	863.95	1155.61	1267.77	1379.93

B.2 Effect of Soaking on Strength values of Stabilized LOC

Samples at 23% moisture content

Sample	7 Days	14 Days	28 Days	56 Days	90 Days
4L - 12G - 23%mc (OS)	1940.82	2452.55	2575.51	4055.62	4750
4L - 12G - 23%mc (PSS)	2277.55	2349.49	2670.41	2856.63	4402.55
4L - 12G - 23%mc (CSS)	1853.57	2073.98	2342.86	2535.72	2938.27

Samples at 28% moisture content

Sample	7 Days	14 Days	28 Days	56 Days	90 Days
4L - 12G - 28%mc (OS)	1599.32	1904.08	2365.65	3491.33	4348.22
4L - 12G - 28%mc (PSS)	1863.27	2025.51	2536.23	2709.19	4045.41
4L - 12G - 28%mc (CSS)	1590.82	1636.23	2112.25	2379.08	3368.88

Samples at 33% moisture content

Sample	7 Days	14 Days	28 Days	56 Days	90 Days
4L - 12G - 33%mc (OS)	1161.22	1434.35	1845.58	3083.17	3151.02
4L - 12G - 33%mc (PSS)	1414.29	1616.84	1746.94	2268.88	2973.47
4L - 12G - 33%mc (CSS)	1232.65	1443.88	1723.98	2171.7	2500.51

Samples at 23% moisture content

Sample	7 Days	14 Days	28 Days	56 Days	90 Days
8L - 8G - 23%mc (OS)	1508.68	2202.05	2287.76	3622.45	4240.31
8L - 8G - 23%mc (PSS)	1641.84	1932.15	2423.22	3398.98	3501.53
8L - 8G - 23%mc (CSS)	1557.66	1659.69	2076.03	2567.86	2750.00

Samples at 28% moisture content

Sample	7 Days	14 Days	28 Days	56 Days	90 Days
8L - 8G - 28%mc (OS)	1566.67	1874.15	2064.28	2630.62	3701.54
8L - 8G - 28%mc (PSS)	1374.49	1419.39	2097.45	2894.9	3049.49
8L - 8G - 28%mc (CSS)	1297.96	1370.92	1721.43	2485.21	2693.88

Samples at 33% moisture content

Sample	7 Days	14 Days	28 Days	56 Days	90 Days
8L - 8G - 33%mc (OS)	1078.23	1310.54	1752.04	2110.72	2645.41
8L - 8G - 33%mc (PSS)	1059.19	1079.09	1459.69	2021.43	2347.45
8L - 8G - 33%mc (CSS)	994.39	1112.25	1469.9	1679.08	2113.27

Samples at 23% moisture content

Sample	7 Days	14 Days	28 Days	56 Days	90 Days
16L - 0G - 23%mc (OS)	1301.53	1371.94	1803.58	1870.92	1641.33
16L - 0G - 23%mc (PSS)	180.11	237.25	525.51	637.25	721.43
16L - 0G - 23%mc (CSS)	142.35	102.55	539.8	484.7	494.9

Samples at 28% moisture content

Sample	7 Days	14 Days	28 Days	56 Days	90 Days
16L - 0G - 28%mc (OS)	913.78	998.47	1385.71	1439.8	2105.11
16L - 0G - 28%mc (PSS)	214.29	248.19	486.74	681.64	771.43
16L - 0G - 28%mc (CSS)	131.64	189.8	491.84	665.31	531.13

Samples at 33% moisture content

Sample	7 Days	14 Days	28 Days	56 Days	90 Days
16L - 0G - 33%mc (OS)	504.59	604.59	891.84	1133.16	1623.47
16L - 0G - 33%mc (PSS)	147.45	161.94	297.45	451.02	708.68
16L - 0G - 33%mc (CSS)	88.27	114.80	275.00	451.54	500.51

Appendix C

c.1a Linear Expansion results (unmellowed)

4L - 12G - 23%mc	4L - 12G - 28%mc	4L - 12G - 33%mc	4L - 12G - 38%mc
0.14	0.43	0.31	0.36
2.02	2.06	1.35	1.35
2.91	3.57	2.28	2.34
3.38	4.04	2.65	2.61
3.44	4.05	2.70	2.60
3.48	4.05	2.71	2.59
3.50	4.06	2.72	2.59
3.53	4.08	2.73	2.45
3.54	4.08	2.73	2.46
3.55	4.08	2.73	2.47
3.56	4.08	2.73	2.47
3.55	4.07	2.73	2.46
3.55	4.05	2.73	2.45
3.54	4.04	2.73	2.44
3.54	4.03	2.73	2.43
3.55	4.03	2.73	2.43
3.56	4.04	2.73	2.44
3.56	4.04	2.73	2.45
3.57	4.05	2.73	2.46
3.58	4.06	2.74	2.47
3.59	4.07	2.74	2.47
3.60	4.08	2.75	2.48
3.61	4.08	2.75	2.49
3.61	4.07	2.75	2.48
3.61	4.07	2.75	2.48
3.61	4.07	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48

3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48
3.61	4.06	2.75	2.48

c.1b Linear Expansion results (unmellowed)

8L - 8G - 23%mc	8L - 8G - 28%mc	8L - 8G - 33%mc	8L - 8G - 38%mc
0.19	0.35	0.20	0.11
0.79	1.60	0.87	0.31
1.33	2.66	1.56	0.53
1.77	3.36	2.17	0.63
2.08	3.78	2.55	0.69
2.34	4.01	2.77	0.77
2.51	4.13	2.87	0.82
2.77	4.20	2.94	0.93
3.16	4.29	2.98	1.09
3.60	4.33	3.01	1.16
3.85	4.36	3.02	1.20
4.01	4.39	3.04	1.23
4.13	4.40	3.05	1.25
4.18	4.41	3.05	1.26
4.21	4.42	3.06	1.27
4.22	4.42	3.07	1.28
4.23	4.42	3.07	1.29
4.23	4.43	3.07	1.29
4.23	4.43	3.07	1.29
4.23	4.43	3.07	1.29
4.22	4.43	3.07	1.29
4.21	4.43	3.07	1.29
4.21	4.43	3.07	1.29
4.21	4.43	3.07	1.30
4.21	4.43	3.07	1.30
4.21	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30

4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30
4.20	4.43	3.07	1.30

c.1c Linear Expansion results (unmellowed)

16L - 0G - 23%mc	16L - 0G - 28%mc	16L - 0G - 33%mc	16L - 0G - 38%mc
0.30	0.12	0.17	0.09
1.00	1.05	0.72	0.45
1.62	1.88	1.34	0.80
2.24	2.70	1.91	1.08
2.85	3.43	2.37	1.34
3.48	4.09	2.92	1.58
4.11	4.67	3.38	1.80
4.85	5.33	3.99	2.39
5.70	5.96	4.69	3.14
6.33	6.49	5.23	3.67
6.94	6.97	5.64	4.08
7.43	7.42	6.12	4.49
7.90	7.81	6.50	4.82
8.36	8.21	6.88	5.15
8.76	8.60	7.21	5.46
9.14	8.94	7.54	5.76
9.53	9.28	7.85	6.03
9.89	9.58	8.13	6.22

10.19	9.83	8.32	6.31
10.41	10.09	8.48	6.42
10.66	10.35	8.69	6.51
11.17	10.59	8.98	6.61
11.68	10.83	9.25	6.68
11.80	10.99	9.48	6.74
11.98	11.10	9.72	6.79
12.20	11.20	9.88	6.85
12.46	11.30	10.03	6.90
12.68	11.39	10.19	6.95
12.90	11.48	10.33	6.99
13.12	11.54	10.46	7.03
13.30	11.60	10.57	7.06
13.48	11.69	10.66	7.08
13.64	11.75	10.74	7.10
13.81	11.81	10.80	7.11
13.95	11.87	10.86	7.11
14.05	11.91	10.89	7.12
14.15	11.93	10.90	7.12
14.26	11.95	10.92	7.13
14.34	11.98	10.94	7.13
14.41	12.00	10.95	7.13
14.47	12.00	10.95	7.13
14.52	12.01	10.96	7.14
14.56	12.02	10.97	7.14
14.60	12.03	10.98	7.14
14.63	12.03	10.98	7.14
14.65	12.03	10.98	7.14
14.67	12.03	10.98	7.14
14.69	12.04	10.98	7.14
14.70	12.04	10.98	7.14
14.70	12.04	10.98	7.14

C.2a Linear Expansion Results (Mellowed)

4L - 12G - 23%mc	4L - 12G - 28%mc	4L - 12G - 33%mc
0.00	0.00	0.00
1.33	0.19	0.50
1.69	0.53	0.76
1.78	0.60	0.80
1.85	0.64	0.83
1.87	0.66	0.84
1.99	0.68	0.85
2.40	1.00	1.01

2.44	1.03	1.03
2.56	1.06	1.07
2.79	1.25	1.22
2.98	1.40	1.23
3.19	1.50	1.23
3.39	1.62	1.24
3.54	1.75	1.24
3.61	1.82	1.26
3.65	1.87	1.26
3.69	1.92	1.27
3.71	1.95	1.28
3.72	1.97	1.29
3.73	1.99	1.30
3.74	2.01	1.31
3.76	2.03	1.32
3.76	2.03	1.33
3.77	2.05	1.34
3.78	2.07	1.35
3.78	2.07	1.35
3.79	2.08	1.35
3.80	2.08	1.35
3.80	2.09	1.35
3.81	2.09	1.35
3.81	2.10	1.36
3.81	2.10	1.36
3.82	2.11	1.37
3.82	2.11	1.37
3.82	2.11	1.38
3.82	2.12	1.38
3.82	2.12	1.38
3.82	2.13	1.39
3.83	2.14	1.39
3.83	2.14	1.39
3.83	2.14	1.39
3.83	2.14	1.39
3.83	2.14	1.39
3.83	2.14	1.39
3.83	2.14	1.39
3.83	2.14	1.39
3.83	2.14	1.39
3.83	2.14	1.39
3.83	2.14	1.39

C.2b Linear Expansion Results (Mellowed)

8L - 8G - 23%mc	8L - 8G - 28%mc	8L - 8G - 33%mc
0.00	0.00	0.00
0.55	2.80	2.31
0.82	3.94	3.27
0.97	4.75	4.01
1.21	4.90	4.08
1.34	4.96	4.11
1.68	5.02	4.13
2.56	5.05	4.16
3.29	5.07	4.18
4.31	5.09	4.18
4.75	5.09	4.18
5.05	5.09	4.18
5.34	5.09	4.18
5.56	5.09	4.18
5.74	5.09	4.18
5.86	5.09	4.19
5.94	5.09	4.19
6.00	5.09	4.19
6.05	5.09	4.19
6.06	5.09	4.19
6.09	5.09	4.20
6.09	5.09	4.20
6.10	5.10	4.21
6.11	5.10	4.21
6.12	5.11	4.22
6.13	5.12	4.22
6.13	5.12	4.23
6.14	5.12	4.23
6.15	5.12	4.24
6.15	5.12	4.24
6.15	5.12	4.24
6.15	5.12	4.24
6.16	5.13	4.24
6.16	5.13	4.24
6.17	5.13	4.24
6.17	5.13	4.24
6.17	5.13	4.24
6.17	5.13	4.24
6.17	5.13	4.24
6.17	5.13	4.24

6.17	5.13	4.24
6.17	5.13	4.24
6.17	5.13	4.24
6.17	5.13	4.24
6.17	5.13	4.24
6.17	5.13	4.24
6.17	5.13	4.24
6.17	5.13	4.24
6.17	5.13	4.24
6.17	5.13	4.24

C.2c Linear Expansion Results (Mellowed)

16L - 0G - 23%mc	16L - 0G - 28%mc	16L - 0G - 33%mc
0.00	0.00	0.00
2.53	0.19	2.06
3.46	0.33	2.87
3.96	0.47	3.43
4.09	0.75	3.67
4.21	0.95	3.89
5.32	1.63	4.07
6.32	2.65	5.34
6.54	2.96	5.45
6.86	3.50	5.62
7.46	4.10	6.17
8.19	4.70	6.74
8.90	5.08	7.20
9.27	5.47	7.60
9.69	5.90	7.87
10.15	6.35	8.22
10.56	6.74	8.53
10.87	7.07	8.79
11.35	7.48	9.12
11.73	7.74	9.44
12.07	7.92	9.68
12.41	8.17	9.86
12.75	8.33	10.10
13.03	8.44	10.28
13.22	8.48	10.46
13.35	8.61	10.62
13.54	8.93	10.75
13.64	9.19	10.86
13.72	9.39	10.98

13.83	9.52	11.07
13.96	9.55	11.16
14.09	9.57	11.21
14.16	9.59	11.27
14.26	9.61	11.29
14.30	9.62	11.31
14.34	9.63	11.34
14.39	9.64	11.35
14.44	9.67	11.35
14.48	9.68	11.36
14.52	9.69	11.38
14.53	9.70	11.39
14.54	9.70	11.40
14.54	9.70	11.40
14.54	9.70	11.40
14.54	9.70	11.40
14.54	9.70	11.40
14.54	9.70	11.40
14.54	9.70	11.40
14.54	9.70	11.40
14.54	9.70	11.40

Appendix D

D.1a. UCS values after 10-days complete immersion in water

Sample	7 Days	14 Days	21 Days	28 Days	35 Days	42 Days	49 Days	56 Days	63 Days	70 Days	77 Days	84 Days	90 Days
4L-12G-23%mc	1232.65	1340.83	1358.94	1377.04	1545.67	1714.29	1882.92	2051.54	1986.54	1921.54	1856.54	1791.53	1726.53
4L-12G-28%mc	1195.41	1431.12	1442.35	1453.57	1705.23	1956.89	2208.55	2460.21	2345.62	2231.03	2116.43	2001.84	1887.25
4L-12G-33%mc	954.08	1135.72	1241.33	1346.94	1525.9	1704.85	1883.81	2062.76	1972.96	1883.17	1793.37	1703.58	1613.78

Sample	7 Days	14 Days	21 Days	28 Days	35 Days	42 Days	49 Days	56 Days	63 Days	70 Days	77 Days	84 Days	90 Days
8L-8G-23%mc	1033.17	1320.92	1180.87	1040.82	1200.64	1360.46	1520.28	1680.1	1661.73	1643.37	1625	1606.64	1588.27
8L-8G-28%mc	895.92	1188.27	1247.96	1307.65	1484.18	1660.72	1837.25	2013.78	1945.72	1877.66	1809.59	1741.53	1673.47
8L-8G-33%mc	896.43	969.9	1015.31	1060.72	1196.18	1331.64	1467.1	1602.56	1598.48	1594.4	1590.31	1586.23	1582.15

Sample	7 Days	14 Days	21 Days	28 Days	35 Days	42 Days	49 Days	56 Days	63 Days	70 Days	77 Days	84 Days	90 Days
16L-0G-23%mc	0	67.35	70.92	74.49	133.04	191.59	250.13	308.68	291.43	274.19	256.94	239.7	222.45
16L-0G-28%mc	35.21	95.41	122.71	150	216.84	283.68	350.51	417.35	403.68	390	376.33	362.65	348.98
16L-0G-33%mc	57.14	89.8	114.29	138.78	190.18	241.59	292.99	344.39	362.45	380.51	398.57	416.63	434.69

Sample	7 Days	14 Days	21 Days	28 Days	35 Days	42 Days	49 Days	56 Days	63 Days	70 Days	77 Days	84 Days	90 Days
4L-12G-23%mc	1232.65	1340.83	1358.94	1377.04	1545.67	1714.29	1882.92	2051.54	1986.54	1921.54	1856.54	1791.53	1726.53
8L-8G-23%mc	1033.17	1320.92	1180.87	1040.82	1200.64	1360.46	1520.28	1680.1	1661.73	1643.37	1625	1606.64	1588.27
16L-0G-23%mc	0	67.35	70.92	74.49	133.04	191.59	250.13	308.68	291.43	274.19	256.94	239.7	222.45

Sample	7 Days	14 Days	21 Days	28 Days	35 Days	42 Days	49 Days	56 Days	63 Days	70 Days	77 Days	84 Days	90 Days
4L-12G-28%mc	1195.41	1431.12	1442.35	1453.57	1705.23	1956.89	2208.55	2460.21	2345.62	2231.03	2116.43	2001.84	1887.25
8L-8G-28%mc	895.92	1188.27	1247.96	1307.65	1484.18	1660.72	1837.25	2013.78	1945.72	1877.66	1809.59	1741.53	1673.47
16L-0G-28%mc	35.21	95.41	122.71	150	216.84	283.68	350.51	417.35	403.68	390	376.33	362.65	348.98

Sample	7 Days	14 Days	21 Days	28 Days	35 Days	42 Days	49 Days	56 Days	63 Days	70 Days	77 Days	84 Days	90 Days
4L-12G-33%mc	954.08	1135.72	1241.33	1346.94	1525.9	1704.85	1883.81	2062.76	1972.96	1883.17	1793.37	1703.58	1613.78
8L-8G-33%mc	896.43	969.9	1015.31	1060.72	1196.18	1331.64	1467.1	1602.56	1598.48	1594.4	1590.31	1586.23	1582.15
16L-0G-33%mc	57.14	89.8	114.29	138.78	190.18	241.59	292.99	344.39	362.45	380.51	398.57	416.63	434.69

D. 2a. Durability Index (DI) values of stabilized LOC completely/partially soaked in water for 4-days.

Sample	7days CSS SRI (%)	14days CSS SRI (%)	28days CSS SRI (%)	56days CSS SRI (%)	90days CSS SRI (%)
4L - 12G - 23%mc	95.5	84.56	90.97	62.52	61.86
8L - 8G - 23%mc	103.25	75.37	90.75	70.89	64.85
16L - 0G - 23%mc	10.94	7.47	29.93	25.91	30.15

Sample	7days CSS SRI (%)	14days CSS SRI (%)	28days CSS SRI (%)	56days CSS SRI (%)	90days CSS SRI (%)
4L - 12G - 28%mc	99.47	85.93	89.29	68.14	77.48
8L - 8G - 28%mc	82.85	73.15	83.39	94.47	72.78
16L - 0G - 28%mc	14.41	19.01	35.49	46.21	25.23

Sample	7days CSS SRI (%)	14days CSS SRI (%)	28days CSS SRI (%)	56days CSS SRI (%)	90days CSS SRI (%)
4L - 12G - 33%mc	106.15	100.66	93.41	70.44	79.36
8L - 8G - 33%mc	92.22	84.87	83.9	79.55	79.88
16L - 0G - 33%mc	17.49	18.99	30.84	39.85	30.83

Sample	7days CSS SRI (%)	14days CSS SRI (%)	28days CSS SRI (%)	56days CSS SRI (%)	90days CSS SRI (%)
4L - 12G - 23%mc	95.5	84.56	90.97	62.52	61.86
4L - 12G - 28%mc	99.47	85.93	89.29	68.14	77.48
4L - 12G - 33%mc	106.15	100.66	93.41	70.44	79.36

Sample	7days CSS SRI (%)	14days CSS SRI (%)	28days CSS SRI (%)	56days CSS SRI (%)	90days CSS SRI (%)
8L - 8G - 23%mc	103.25	75.37	90.75	70.89	64.85
8L - 8G - 28%mc	82.85	73.15	83.39	94.47	72.78
8L - 8G - 33%mc	92.22	84.87	83.9	79.55	79.88

Sample	7days CSS SRI (%)	14days CSS SRI (%)	28days CSS SRI (%)	56days CSS SRI (%)	90days CSS SRI (%)
16L - 0G - 23%mc	10.94	7.47	29.93	25.91	30.15
16L - 0G - 28%mc	14.41	19.01	35.49	46.21	25.23
16L - 0G - 33%mc	17.49	18.99	30.84	39.85	30.83

Sample	7days PSS SRI (%)	14days PSS SRI (%)	28days PSS SRI (%)	56days PSS SRI (%)	90days PSS SRI (%)
4L - 12G - 23%mc	117.35	95.8	103.68	70.44	92.69
8L - 8G - 23%mc	108.83	87.74	105.92	93.83	82.58
16L - 0G - 23%mc	13.84	17.29	29.14	34.06	43.95

Sample	7days PSS SRI (%)	14days PSS SRI (%)	28days PSS SRI (%)	56days PSS SRI (%)	90days PSS SRI (%)
4L - 12G - 28%mc	116.5	106.38	107.21	77.6	93.04
8L - 8G - 28%mc	87.73	75.74	101.61	110.05	82.38
16L - 0G - 28%mc	23.45	24.86	35.13	47.34	36.65

Sample	7days PSS SRI (%)	14days PSS SRI (%)	28days PSS SRI (%)	56days PSS SRI (%)	90days PSS SRI (%)
4L - 12G - 33%mc	121.79	112.72	94.66	73.59	94.37
8L - 8G - 33%mc	98.23	82.34	83.31	95.77	88.74

16L - 0G - 33%mc	29.22	26.79	33.35	39.8	43.65

Sample	7days PSS SRI (%)	14days PSS SRI (%)	28days PSS SRI (%)	56days PSS SRI (%)	90days PSS SRI (%)
4L - 12G - 23%mc	117.35	95.8	103.68	70.44	92.69
4L - 12G - 28%mc	116.5	106.38	107.21	77.6	93.04
4L - 12G - 33%mc	121.79	112.72	94.66	73.59	94.37

Sample	7days PSS SRI (%)	14days PSS SRI (%)	28days PSS SRI (%)	56days PSS SRI (%)	90days PSS SRI (%)
8L - 8G - 23%mc	108.83	87.74	105.92	93.83	82.58
8L - 8G - 28%mc	87.73	75.74	101.61	110.05	82.38
8L - 8G - 33%mc	98.23	82.34	83.31	95.77	88.74

Sample	7days PSS SRI (%)	14days PSS SRI (%)	28days PSS SRI (%)	56days PSS SRI (%)	90days PSS SRI (%)
16L - 0G - 23%mc	13.84	17.29	29.14	34.06	43.95
16L - 0G - 28%mc	23.45	24.86	35.13	47.34	36.65
16L - 0G - 33%mc	29.22	26.79	33.35	39.8	43.65

D.2b. Strength Durability index values after 10 days soaking at varying curing ages

Sample	7 Days	14 Days	28 Days	56 Days	90 Days
4L-12G-23%mc	63.51	54.67	53.47	50.59	36.35
4L-12G-28%mc	74.74	75.16	61.44	70.47	43.4
4L-12G-33%mc	82.16	79.18	72.98	66.9	51.21

Sample	7 Days	14 Days	28 Days	56 Days	90 Days
8L-8G-23%mc	68.48	59.99	45.5	46.38	37.46
8L-8G-28%mc	57.19	63.4	63.35	76.55	45.21
8L-8G-33%mc	83.14	74.01	60.54	75.92	59.81

Sample	7 Days	14 Days	28 Days	56 Days	90 Days
16L-0G-23%mc	0	4.91	4.13	16.5	13.55
16L-0G-28%mc	3.85	9.56	10.82	28.99	16.58
16L-0G-33%mc	11.32	14.85	15.56	30.39	26.78

Sample	7 Days	14 Days	28 Days	56 Days	90 Days
4L-12G-23%mc	63.51	54.67	53.47	50.59	36.35
8L-8G-23%mc	68.48	59.99	45.5	46.38	37.46
16L-0G-23%mc	0	4.91	4.13	16.5	13.55

Sample	7 Days	14 Days	28 Days	56 Days	90 Days
4L-12G-28%mc	74.74	75.16	61.44	70.47	43.4
8L-8G-28%mc	57.19	63.4	63.35	76.55	45.21
16L-0G-28%mc	3.85	9.56	10.82	28.99	16.58

Sample	7 Days	14 Days	28 Days	56 Days	90 Days
4L-12G-33%mc	82.16	79.18	72.98	66.9	51.21
8L-8G-33%mc	83.14	74.01	60.54	75.92	59.81
16L-0G-33%mc	11.32	14.85	15.56	30.39	26.78

D. 3a. Water absorption profile of stabilized Lower Oxford Clay after 7-days curing and 24-hours soaking.

7 Days Curing Age

Sample	Weight percent of Water absorbed after											
	2	4	6	8	10	12	14	16	18	20	22	24
4L - 12G - 23%mc	1.44	2.02	2.60	2.84	3.08	3.31	3.55	3.79	4.27	4.50	4.74	4.75
4L - 12G - 28%mc	0.38	0.58	0.77	0.93	1.09	1.25	1.41	1.58	1.74	1.90	2.06	2.22
4L - 12G - 33%mc	0.26	0.42	0.57	0.66	0.76	0.85	0.94	1.04	1.13	1.22	1.31	1.41

7 Days Curing Age

Sample	Weight percent of Water absorbed after											
	2	4	6	8	10	12	14	16	18	20	22	24
8L - 8G - 23%mc	3.63	4.12	4.60	4.71	4.81	4.92	5.02	5.13	5.24	5.34	5.45	5.56
8L - 8G - 28%mc	0.79	1.11	1.43	1.58	1.73	1.89	2.04	2.19	2.34	2.49	2.65	2.80
8L - 8G - 33%mc	0.51	0.73	0.94	1.06	1.18	1.30	1.42	1.55	1.67	1.79	1.91	2.03

7 Days Curing Age

Sample	Weight of percent Water absorbed after											
	2	4	6	8	10	12	14	16	18	20	22	24
16L - 0G - 23%mc	9.96	10.18	10.39	10.53	10.67	10.81	10.95	11.10	11.24	11.38	11.52	11.66
16L - 0G - 28%mc	4.70	4.87	5.04	5.17	5.31	5.44	5.58	5.71	5.84	5.98	6.11	6.25
16L - 0G - 33%mc	2.70	2.89	3.08	3.21	3.34	3.46	3.59	3.72	3.85	3.97	4.10	4.23

7 Days Curing Age

Sample	Weight percent of Water absorbed after											
	2	4	6	8	10	12	14	16	18	20	22	24
4L - 12G - 23%mc	1.44	2.02	2.60	2.84	3.08	3.31	3.55	3.79	4.27	4.50	4.74	4.75
8L - 8G - 23%mc	3.63	4.12	4.60	4.71	4.81	4.92	5.02	5.13	5.24	5.34	5.45	5.56
16L - 0G - 23%mc	9.96	10.18	10.39	10.53	10.67	10.81	10.95	11.10	11.24	11.38	11.52	11.66

7 Days Curing Age

Sample	Weight percent of Water absorbed after											
	2	4	6	8	10	12	14	16	18	20	22	24
4L - 12G - 28%mc	0.38	0.58	0.77	0.93	1.09	1.25	1.41	1.58	1.74	1.90	2.06	2.22
8L - 8G - 28%mc	0.79	1.11	1.43	1.58	1.73	1.89	2.04	2.19	2.34	2.49	2.65	2.80
16L - 0G - 28%mc	4.70	4.87	5.04	5.17	5.31	5.44	5.58	5.71	5.84	5.98	6.11	6.25

7 Days Curing Age

Sample	Weight percent of Water absorbed after											
	2	4	6	8	10	12	14	16	18	20	22	24
4L - 12G - 33%mc	0.26	0.42	0.57	0.66	0.76	0.85	0.94	1.04	1.13	1.22	1.31	1.41
8L - 8G - 33%mc	0.51	0.73	0.94	1.06	1.18	1.30	1.42	1.55	1.67	1.79	1.91	2.03
16L - 0G - 33%mc	2.70	2.89	3.08	3.21	3.34	3.46	3.59	3.72	3.85	3.97	4.10	4.23

D. 3b. Water absorption profile of stabilized Lower Oxford Clay after 7-days curing and 10-days soaking.

7 Days Curing Age

Sample	Weight percent of water absorbed after					
	1	2	4	6	8	10
4L - 12G - 23%mc	4.75	5.55	6.14	6.20	6.27	6.33
4L - 12G - 28%mc	2.22	3.00	3.39	3.50	3.62	3.73
4L - 12G - 33%mc	1.41	1.89	2.14	2.18	2.23	2.27

7 Days Curing Age

Sample	Weight percent of water absorbed after					
	1	2	4	6	8	10
8L - 8G - 23%mc	5.56	6.15	6.80	6.88	6.96	7.04
8L - 8G - 28%mc	2.80	3.36	3.86	3.93	4.01	4.08
8L - 8G - 33%mc	2.03	2.45	2.76	2.83	2.90	2.97

7 Days Curing Age

Sample	Weight percent of water absorbed after					
	1	2	4	6	8	10
16L - 0G - 23%mc	11.66	13.22	13.73	14.94	16.15	17.36

16L - 0G - 28%mc	6.25	7.67	9.61	11.49	13.37	15.26
16L - 0G - 33%mc	4.23	5.56	7.09	8.13	9.17	10.22

7 Days Curing Age

Sample	Weight percent of water absorbed after					
	1	2	4	6	8	10
4L - 12G - 23%mc	4.75	5.55	6.14	6.20	6.27	6.33
8L - 8G - 23%mc	5.56	6.15	6.80	6.88	6.96	7.04
16L - 0G - 23%mc	11.66	13.22	13.73	14.94	16.15	17.36

7 Days Curing Age

Sample	Weight percent of water absorbed after					
	1	2	4	6	8	10
4L - 12G - 28%mc	2.22	3.00	3.39	3.50	3.62	3.73
8L - 8G - 28%mc	2.80	3.36	3.86	3.93	4.01	4.08
16L - 0G - 28%mc	6.25	7.67	9.61	11.49	13.37	15.26

7 Days Curing Age

Sample	Weight percent of water absorbed after					
	1	2	4	6	8	10
4L - 12G - 33%mc	1.41	1.89	2.14	2.18	2.23	2.27
8L - 8G - 33%mc	2.03	2.45	2.76	2.83	2.90	2.97

16L - 0G - 33%mc	4.23	5.56	7.09	8.13	9.17	10.22

D. 3c. Water absorption profile of stabilized Lower Oxford Clay after 14-days curing and 10-days soaking.

14-Days Curing

Sample	Weight percent of water absorbed after					
	1	2	4	6	8	10
4L - 12G - 23%mc	4.36	5.38	6.03	6.14	6.25	6.36
4L - 12G - 28%mc	1.6	2.43	2.94	3.03	3.12	3.22
4L - 12G - 33%mc	0.97	1.47	1.84	1.92	1.99	2.07

14-Days Curing

Sample	Weight percent of water absorbed after					
	1	2	4	6	8	10
8L - 8G - 23%mc	4.65	5.38	5.95	6.04	6.13	6.22
8L - 8G - 28%mc	2.02	2.69	3.21	3.32	3.43	3.54
8L - 8G - 33%mc	1.46	2.04	2.33	2.42	2.52	2.61

14-Days Curing

Sample	Weight percent of water absorbed after					
	1	2	4	6	8	10
16L - 0G - 23%mc	9.48	10.42	11.63	12.65	13.67	14.7
16L - 0G - 28%mc	5.06	5.67	6.99	8.08	9.16	10.25
16L - 0G - 33%mc	3.67	4.53	5.82	6.75	7.68	8.61

14-Days Curing

Sample	Weight percent of water absorbed after					
	1	2	4	6	8	10
4L - 12G - 23%mc	4.36	5.38	6.03	6.14	6.25	6.36
8L - 8G - 23%mc	4.65	5.38	5.95	6.04	6.13	6.22
16L - 0G - 23%mc	9.48	10.42	11.63	12.65	13.67	14.7

14-Days Curing

Sample	Weight percent of water absorbed after					
	1	2	4	6	8	10
4L - 12G - 28%mc	1.6	2.43	2.94	3.03	3.12	3.22
8L - 8G - 28%mc	2.02	2.69	3.21	3.32	3.43	3.54
16L - 0G - 28%mc	5.06	5.67	6.99	8.08	9.16	10.25

14-Days Curing

Sample	Weight percent of water absorbed after					
	1	2	4	6	8	10
4L - 12G - 33%mc	0.97	1.47	1.84	1.92	1.99	2.07
8L - 8G - 33%mc	1.46	2.04	2.33	2.42	2.52	2.61
16L - 0G - 33%mc	3.67	4.53	5.82	6.75	7.68	8.61

Appendix E1 Global flooding events register of year 2003 (http://www.dartmouth.edu/~floods/DataProducts/ExtremeFloods/examples.html).

Location	Continent	Detailed Locations	Notes and Comments
W. Kenya	Africa	Kisumu - Manyattta, Dunga and Nyalenda. Kisii. River Auji.	Affected 390 sq km area and lasted for 2 days.
Germany, Portugal, Belgium, Romania	Europe	Germany - Towns: Coburg, Zell, Cologne, Koblenz, Regensburg, Wertheim, Soemmerda. Rivers: Moselle, Main, Saale, Itz, Rhine. Central and northern Portugal - Agueda river in Agueda town. Douro River. Bairrada region. Romania - Bistrita river near Poiana Teiului Southern and western Belgium - Meuse River. Dender River at Geraardsbergen. Western city of Ghent, towns and villages southwest of Brussels. Northern France - Lisieux Netherlands - Meuse River Czech Republic - Mrlina river at Vestec. Vltava River near Prague.	A week of heavy rain in Europe swells rivers. Several Rhine tributaries have broken banks in Germany. Flooding and landslides in Portugal. Bistrita river in Romania floods hundreds of houses in four villages. Worst floods in a nearly a decade in Belgium. Affected 167000 sq km area and lasted for 5 days.
UK	Europe	UK - Southern England. Anglia region. Thames valley. Midlands. Sussex, Kent and Hampshire. Tributaries of the Medway river at Yalding in Kent. Helston and Crowlas in Cornwall. Paignton, Torquay and Lympstone in Devon. River Avon in Malmesbury, Wiltshire. Thames Ditton Island. Chertsey in Surrey.	January 3 - Over 399 flood warnings in effect in England and Wales, the ground is saturated. "'The rivers are right up to the tops of the banks and they just cannot take any more." "River Thames had reached "extremely high levels", with flows in some places higher than those experienced during the 'great flood of 1947'" Affected 23090 sq km area and lasted for 4 days.
Southern US	North America	Texas - Rivers: Sabine, Angelina, Guadalupe, Navasota, Neches, Nueces, Sabine, Sulphur; Attoyac and Black Cypress Bayous; Little Cypress Creek. Western Florida - Rivers: Withlacoochee, Alafia, Hillsborough, Little Manatee, Manatee, Peace. Cypress Creek. Mississippi - Rivers: Pearl, Big Sunflower, Big Black, Louisiana - Rivers: Calcasieu, Little	Jan 3 - USGS flood gauges show several rivers above flood stage in western Florida, eastern Texas, Lousiana. Mississippi, Arkansas. Flooding is minor. Affected 144000 sq km area and lasted for 3 days.
Congo and Zaire	Africa	North and Central Congo - Lower Congo River and Tributaries including Likouala aux Herbes, Oubangui, Giri, Sangha, Ngoko. Loukolelas, Makotimpoko and Mossaka	Detected by monitoring MODIS imagery in early November - flooding still observed in early January 2003 "Floods caused by torrential rain and the swelling of the Congo river in recent weeks have left thousands of people without shelter in central and northern parts of Congo" Affected 115400 sq km area and lasted for 57 days.

Appendix E 2.

Examples of major floods (http://www.lenntech.com/flood.htm)

Flooding occurs all over the world. The examples of major floods mentioned below illustrate this.

- In November 2006, major floods hit large parts of Somalia, killing at least 52 people and displacing 50,000 others. The government appealed for international help to prevent a humanitarian catastrophe
- In November 2006, heavy rain caused large parts of Afghanistan to flood. The official death toll was set on 80, and it took more than two days to reach some of the most severely flooded villages
- In August 2006, a major part of Ethiopia was flooded
- Widespread flood of the Mid-Atlantic region of the eastern United States in 2006. At least 16 deaths. Delaware, Columbia, New Jersey, Maryland, Pennsylvania, Virginia, and New York are stricken.
- In May 2006, a major part of Korea was flooded
- In April 2006, the Danube River flooded large parts of Budapest, Hungary. Margit Island was completely under water for a significant amount of time
- In November 2005, some Indian states in the Bay of Bengal were isolated due to heavy rainfall
- In August 2005, major parts of Eastern Europe were flooded due to heavy rainfall
- In August 2005, hurricane Katrina hit New Orleans, the hurricane destroyed levees and floodwalls in the river area, causing 80% of the town to be flooded. Some parts were more than six metres under water. More than 80% of the residents were evacuated, and many were housed in the Louisiana Superdome last-minute. The official death toll from the flood was set on 1,577. Economic and political damage lasted well into 2006
- In July 2005, parts of Mumbai India were severely flooded. In some areas the water was more than 4.5 metres high. The flood cost over 700 lives
- In June 2005, Alberta, Canada was flooded, lasting three weeks and demanding 72 lives
- In 2000 large parts of Mozambique were flooded after a cyclone, killing thousands of people
- In the summer of 1998, China experienced major floods of the Yangtze River, killing more than 3,000 people, and leaving 14 million homeless
- In 1994 and 1995 heavy rainfall caused Rivers to flood parts of The Netherlands (see picture). Many people in the province of Limburg were evacuated, and dikes in other provinces threatened to break through
- In 1993 the Great Flood hit the American Midwest. The official death toll was 28, and 10,000 homes were destroyed along with 15 million acres of farmland
- In January 1992 large parts of South America flooded
- In 1965 hurricane Betsy flooded major parts of New Orleans, killing 40 people
- In 1957 about 400 people were killed in Louisiana by flooding caused by hurricane Audrey
- In 1955 the Australian Hunter Valley was flooded due to heavy rainfall. In total 25 people were killed, 2,000 cattle drowned, and millions of dollars of crop were destroyed
- In 1953 many Dutch and British citizens were killed in the North Sea flood. For an extensive description, see environmental disasters
- In 1952 a flood of Lynmouth, England, killed 34 people and destroyed over 80 buildings
- In 1951 heavy rains caused a flood in the Kansas River area, which cost 28 lives. Large parts of Kansas flooded, and damages amounted up to almost one billion dollars

- In 1937 the Ohio River flooded, damaging Pittsburgh and Cairo, Illinois. Approximately 385 people died and about one million were left homeless
- In 1931 major parts of China flooded, causing between 800,000 and 4,000,000 casualties
- In 1927 the United States experienced the Great Mississippi flood, as a result of heavy rainfall. The flood affected at least 7 different states, killing 246 people and displacing more than 700,000

Appendix F

Regression Analysis

Welcome to Minitab, press F1 for help.
Executing from file: C:\Program Files\Minitab 15\English\Macros\Startup.mac

This Software was purchased for academic use only.
Commercial use of the Software is prohibited.

Strength

Correlations: Curing Age (Days), Compaction m/c (%)

Pearson correlation of Curing Age (Days) and Compaction m/c (%) = 0.000
P-Value = 1.000

Correlations: UCS kN/m^2, Curing Age (Days)

Pearson correlation of UCS kN/m^2 and Curing Age (Days) = 0.272
P-Value = 0.015

Regression Analysis: UCS kN/m^2 versus Curing Age (, Compaction m, ...

* Compaction Density (Mg/m^3) is (essentially) constant
* Compaction Density (Mg/m^3) has been removed from the equation.

The regression equation is
UCS kN/m^2 = 493 + 15.6 Curing Age (Days) - 1.85 Compaction m/c (%)

Predictor	Coef	SE Coef	T	P
Constant	493.1	294.2	1.68	0.098
Curing Age (Days)	15.578	6.285	2.48	0.015
Compaction m/c (%)	-1.851	8.799	-0.21	0.834

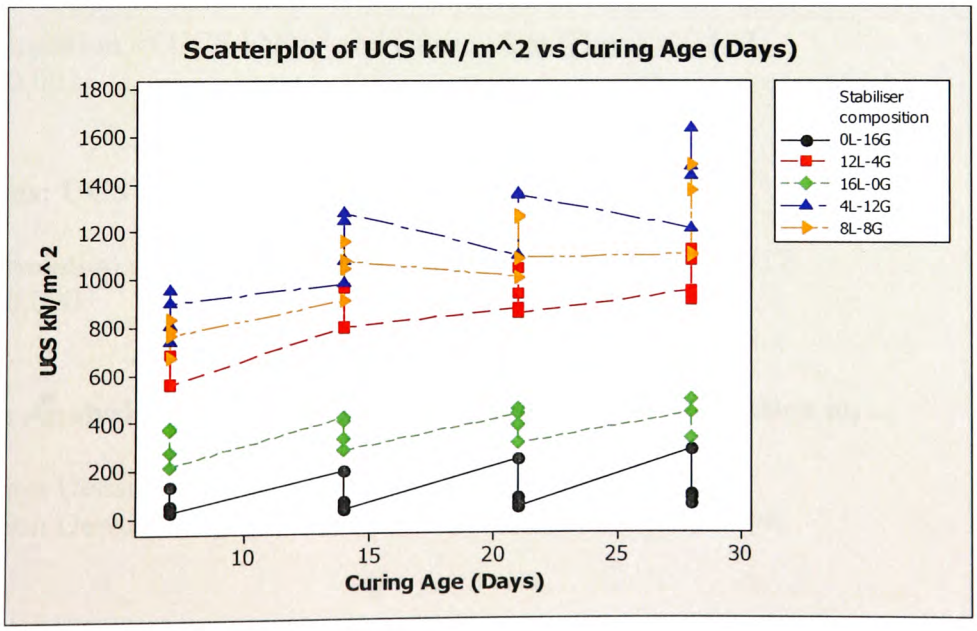
S = 439.940 R-Sq = 7.4% R-Sq(adj) = 5.0%

Analysis of Variance

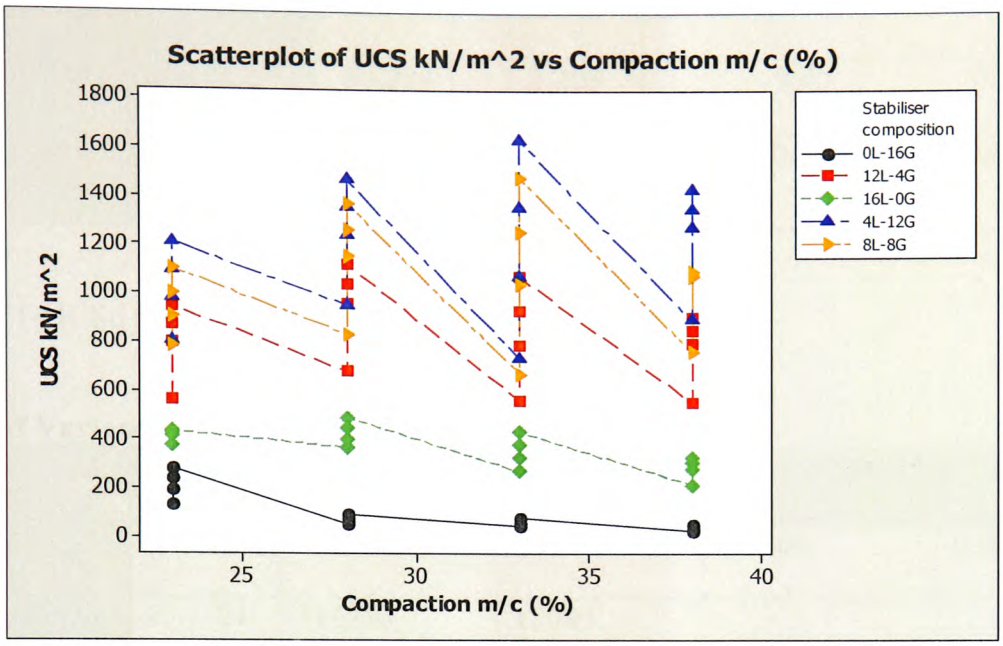
Source	DF	SS	MS	F	P
Regression	2	1197603	598802	3.09	0.051
Residual Error	77	14903158	193548		
Total	79	16100761			

Source	DF	Seq SS
Curing Age (Days)	1	1189035
Compaction m/c (%)	1	8568

Scatterplot of UCS kN/m^2 vs Curing Age (Days)



Scatterplot of UCS kN/m^2 vs Compaction m/c (%)



Results for: Worksheet 2

Correlations: UCS kN/m^2, Curing Age (Days)

Pearson correlation of UCS kN/m^2 and Curing Age (Days) = 0.627
P-Value = 0.001

Correlations: UCS kN/m^2, Compaction m/c (%)

Pearson correlation of UCS kN/m^2 and Compaction m/c (%) = -0.717
P-Value = 0.000

Regression Analysis: UCS kN/m^2 versus Curing Age (, Compaction m, ...

* Compaction Density (Mg/m^3) is (essentially) constant
* Compaction Density (Mg/m^3) has been removed from the equation.

The regression equation is
UCS kN/m^2 = 3218 + 30.3 Curing Age (Days) - 66.4 Compaction m/c (%)

Predictor	Coef	SE Coef	T	P
Constant	3218.1	211.8	15.20	0.000
Curing Age (Days)	30.284	3.203	9.45	0.000
Compaction m/c (%)	-66.355	6.141	-10.81	0.000

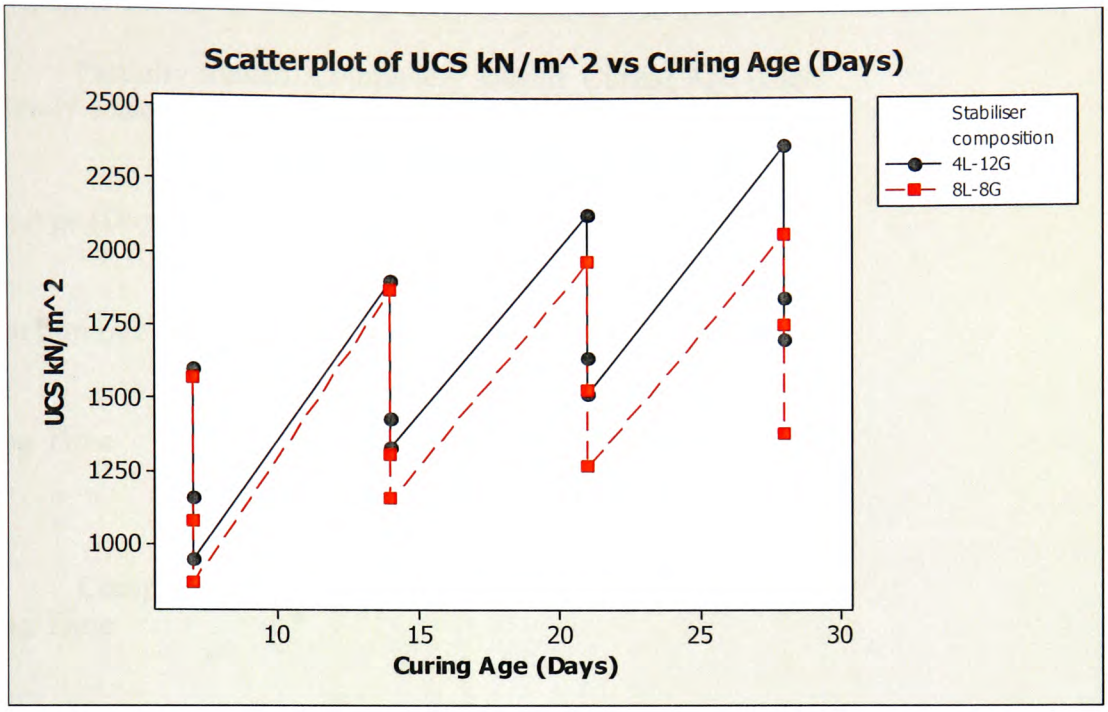
S = 122.821 R-Sq = 90.8% R-Sq(adj) = 89.9%

Analysis of Variance

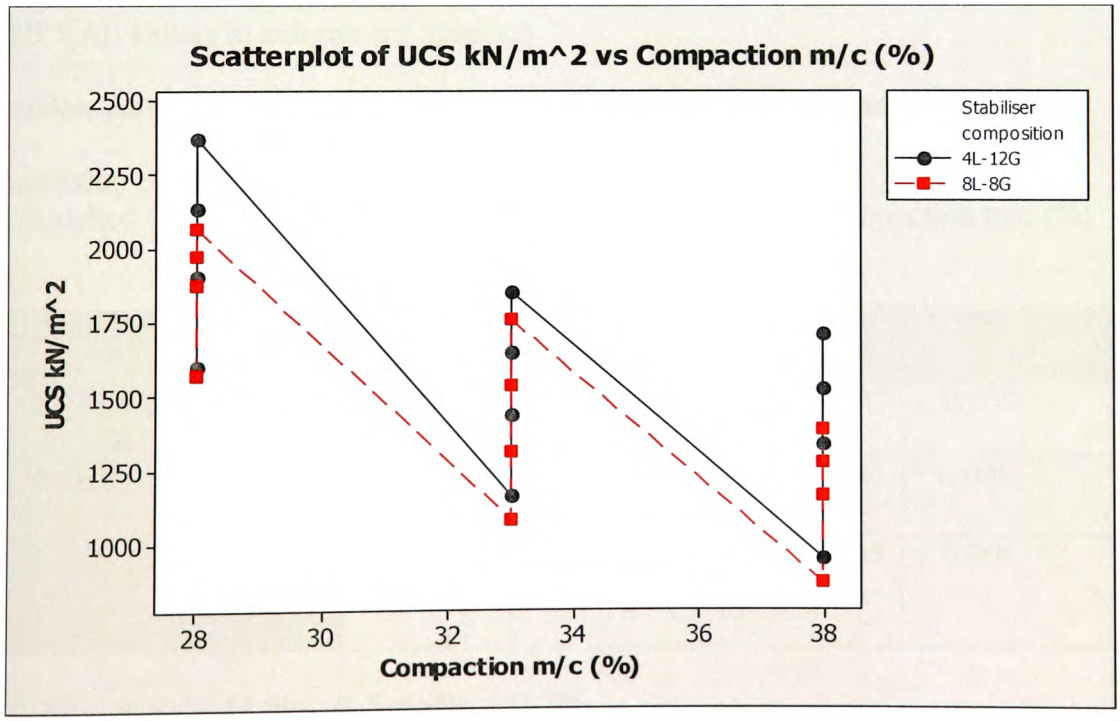
Source	DF	SS	MS	F	P
Regression	2	3109388	1554694	103.06	0.000
Residual Error	21	316783	15085		
Total	23	3426171			

Source	DF	Seq SS
Curing Age (Days)	1	1348174
Compaction m/c (%)	1	1761214

Scatterplot of UCS kN/m^2 vs Curing Age (Days)



Scatterplot of UCS kN/m^2 vs Compaction m/c (%)



Soaked Strength

Correlations: Partially so, Completely s, Curing Age (, Compaction m, ...

	Partially soaked	Completely soaked	Curing Age (Days
Completely soaked	0.985		
	0.000		
Curing Age (Days	0.452	0.406	
	0.002	0.006	
Compaction m/c (	-0.243	-0.187	0.000
	0.107	0.219	1.000
Soaking Time	*	*	*
	*	*	*

	Compaction m/c (
Soaking Time	*
	*

Cell Contents: Pearson correlation
P-Value

* NOTE * All values in column are identical.

Regression Analysis: UCS Unsoaked versus Curing Age (, Compaction m

The regression equation is
UCS Unsoaked (kN/m^2) = 3717 + 22.1 Curing Age (Days) - 88.6 Compaction m/c (%)

Predictor	Coef	SE Coef	T	P
Constant	3717.1	738.4	5.03	0.000
Curing Age (Days)	22.081	3.432	6.43	0.000
Compaction m/c (%)	-88.63	25.66	-3.45	0.001

S = 702.776 R-Sq = 55.9% R-Sq(adj) = 53.8%

Analysis of Variance

Source	DF	SS	MS	F	P
Regression	2	26340181	13170090	26.67	0.000
Residual Error	42	20743560	493894		
Total	44	47083741			

Source	DF	Seq SS
Curing Age (Days)	1	20448174
Compaction m/c (%)	1	5892007

Unusual Observations

Obs	Curing Age (Days)	UCS Unsoaked (kN/m^2)	Fit	SE Fit	Residual	St Resid
43	90.0	1641	3666	241	-2024	-3.07R

R denotes an observation with a large standardized residual.

Regression Analysis: Partially so versus Curing Age (, Compaction m

The regression equation is
Partially soaked (kN/m^2) = 2909 + 16.6 Curing Age (Days) - 66.7 Compaction m/c (%)

Predictor	Coef	SE Coef	T	P
Constant	2909	1044	2.79	0.008
Curing Age (Days)	16.554	4.851	3.41	0.001
Compaction m/c (%)	-66.68	36.28	-1.84	0.073

S = 993.544 R-Sq = 26.3% R-Sq(adj) = 22.8%

Analysis of Variance

Source	DF	SS	MS	F	P
Regression	2	14828138	7414069	7.51	0.002
Residual Error	42	41459418	987129		
Total	44	56287556			

Source	DF	Seq SS
Curing Age (Days)	1	11493411
Compaction m/c (%)	1	3334727

Unusual Observations

Obs	Curing Age (Days)	Partially soaked (kN/m^2)	Fit	SE Fit	Residual	St Resid
43	90.0	721	2865	341	-2144	-2.30R

R denotes an observation with a large standardized residual.

* NOTE * All values in column are identical.

Regression Analysis: Completely s versus Soaking Time, Compaction m, ...

* Soaking Time is (essentially) constant
* Soaking Time has been removed from the equation.

The regression equation is
Completely soaked (kN/m^2) = 2131 - 41.7 Compaction m/c (%) + 12.1 Curing Age (Days)

Predictor	Coef	SE Coef	T	P
Constant	2131.5	885.4	2.41	0.021
Compaction m/c (%)	-41.65	30.77	-1.35	0.183
Curing Age (Days)	12.095	4.115	2.94	0.005

S = 842.699 R-Sq = 20.0% R-Sq(adj) = 16.1%

Analysis of Variance

Source	DF	SS	MS	F	P
Regression	2	7436543	3718272	5.24	0.009
Residual Error	42	29825958	710142		
Total	44	37262501			

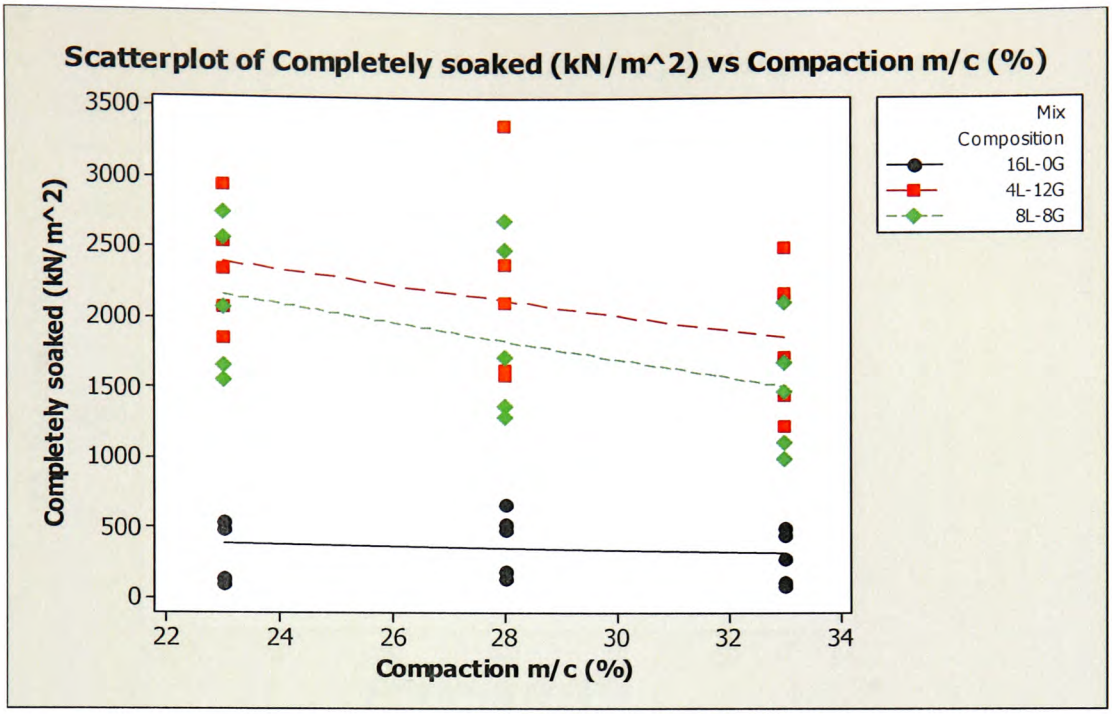
Source	DF	Seq SS
Compaction m/c (%)	1	1301338
Curing Age (Days)	1	6135205

Unusual Observations

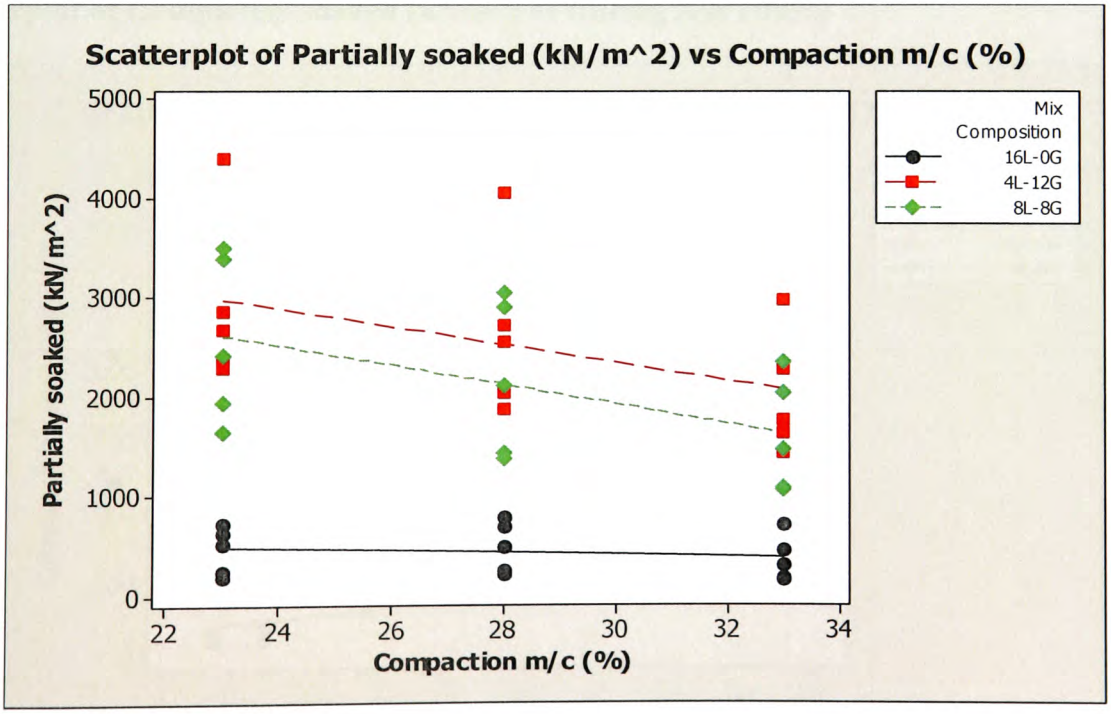
Obs	Compaction m/c (%)	Completely soaked (kN/m^2)	Fit	SE Fit	Residual	St Resid
43	23.0	495	2262	289	-1767	-2.23R

R denotes an observation with a large standardized residual.

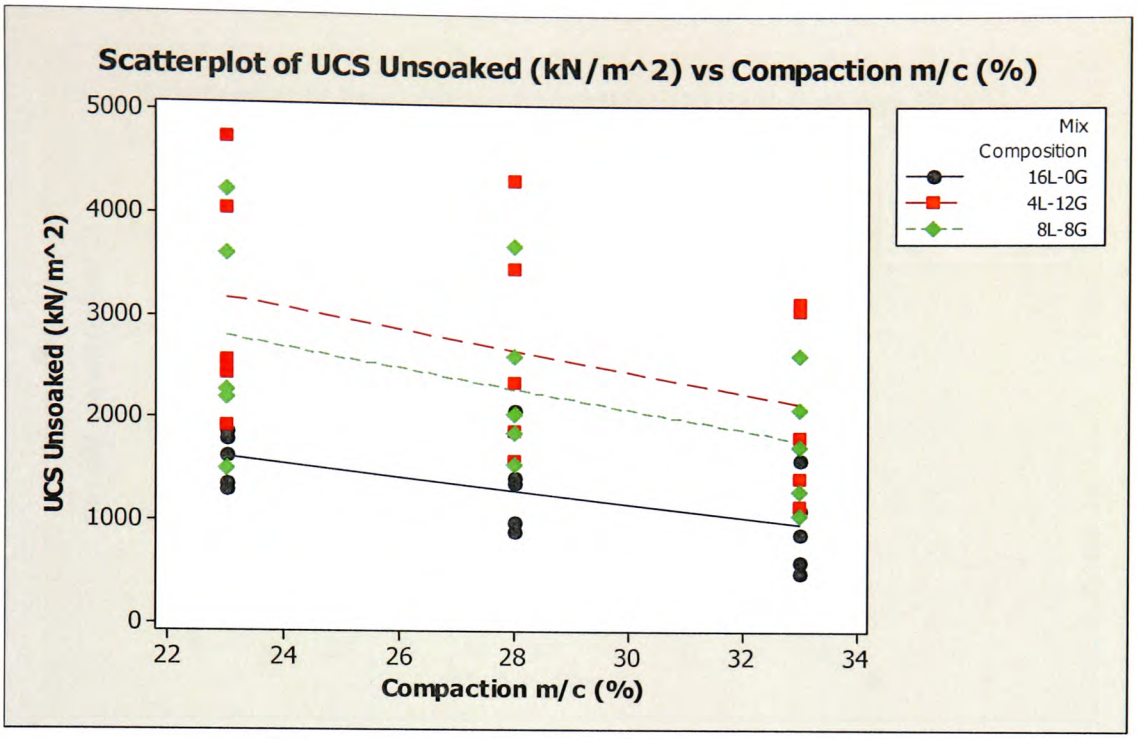
Scatterplot of Completely soaked (kN/m^2) vs Compaction m/c (%)



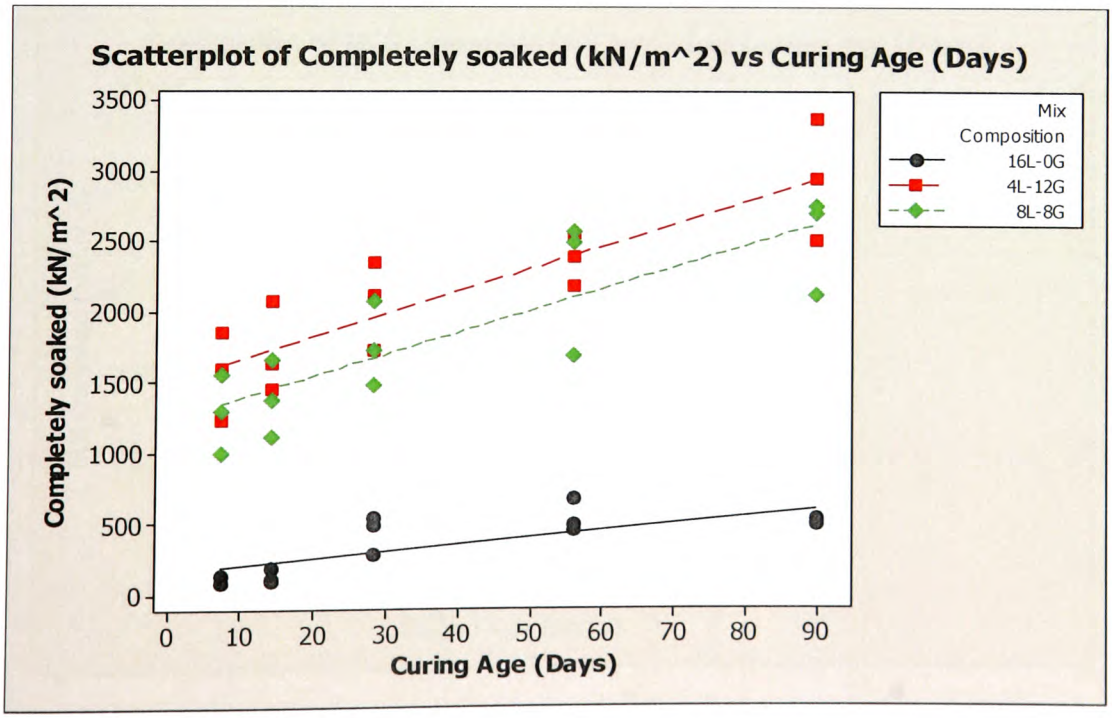
Scatterplot of Partially soaked (kN/m^2) vs Compaction m/c (%)



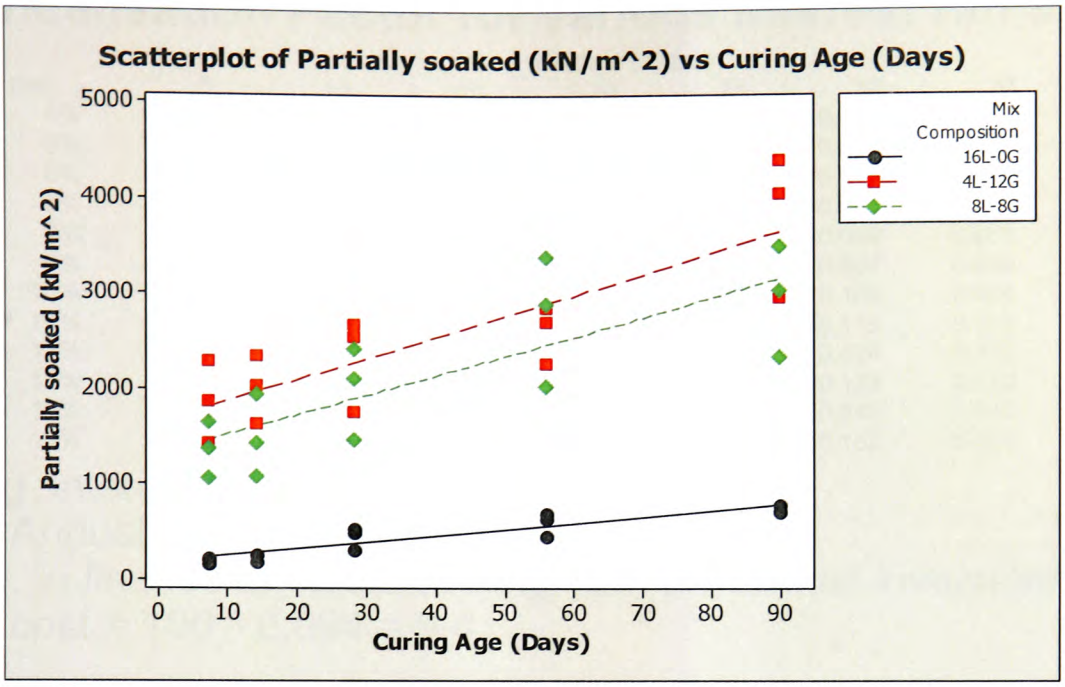
Scatterplot of UCS Unsoaked (kN/m^2) vs Compaction m/c (%)



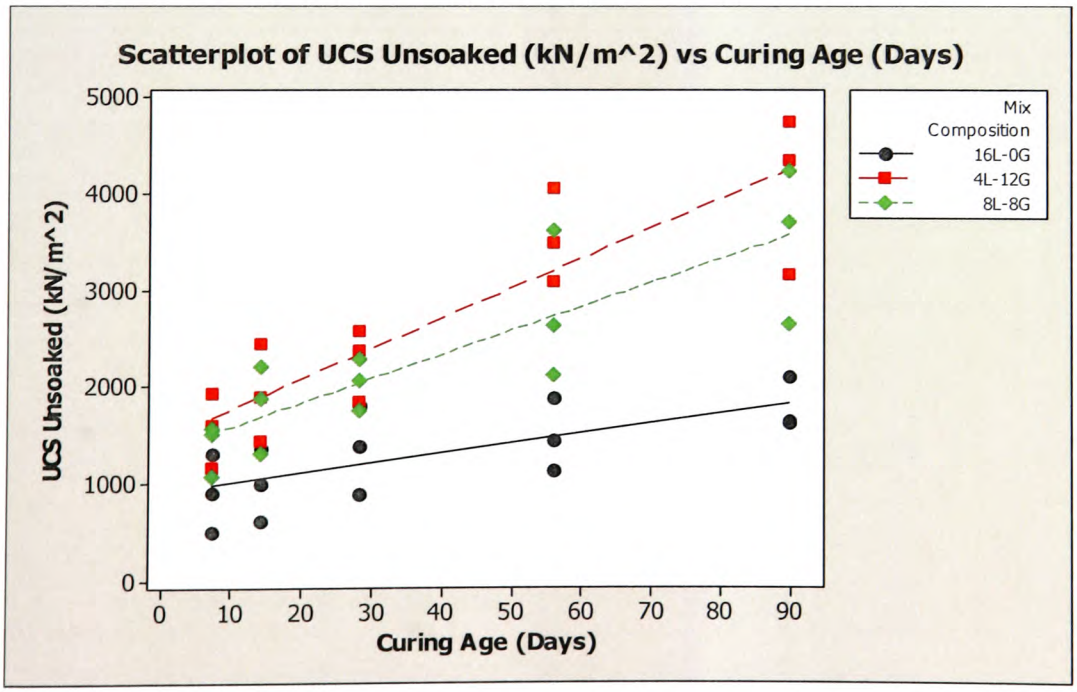
Scatterplot of Completely soaked (kN/m^2) vs Curing Age (Days)



Scatterplot of Partially soaked (kN/m^2) vs Curing Age (Days)



Scatterplot of UCS Unsoaked (kN/m^2) vs Curing Age (Days)



Appendix F1 Annualisation factors

Annualisation Factor for various interest rates (1)

	Lifetime (Years)							
Interest rate	5	10	15	20	25	30	35	40
4%	0.225	0.123	0.090	0.074	0.064	0.058	0.054	0.051
5%	0.231	0.130	0.096	0.080	0.071	0.065	0.061	0.058
6%	0.237	0.136	0.103	0.087	0.078	0.073	0.069	0.066
7%	0.244	0.142	0.110	0.094	0.086	0.081	0.077	0.075
8%	0.250	0.149	0.117	0.102	0.094	0.089	0.086	0.084
9%	0.257	0.156	0.124	0.110	0.102	0.097	0.095	0.093
10%	0.264	0.163	0.131	0.117	0.110	0.106	0.104	0.102
11%	0.271	0.170	0.139	0.126	0.119	0.115	0.113	0.112
12%	0.277	0.177	0.147	0.134	0.127	0.124	0.122	0.121
13%	0.284	0.184	0.155	0.142	0.136	0.133	0.132	0.131
14%	0.291	0.192	0.163	0.151	0.145	0.143	0.141	0.141
15%	0.298	0.199	0.171	0.160	0.155	0.152	0.151	0.151

E.g. interest rate = 7%, lifetime = 20 years
Annualisation factor = 0.094
I.e. to finance an asset costing 100, the annual investment cost = 100 * 0.094 = 9.4

*Extracted from Investment analysis for Power Quality Solutions

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Enhancing the durability of flooded low-capacity soils by utilizing lime-activated ground granulated blastfurnace slag (GGBS)

G.N. Obuzor *, J.M. Kinuthia, R.B. Robinson

Civil Engineering Research Unit, Department of Engineering, Faculty of Advanced Technology, University of Glamorgan, Pontypridd, CF37 1DL, UK

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ABSTRACT

Flooding has a significant impact on road infrastructural development and maintenance. This study was aimed at mitigating the effects of flooding of low-bearing capacity soils, a solution that could alleviate the effects of flooding on road structural layers, especially, those constructed on flood susceptible soils and/or regions. The achievement of such an improvement would foster the growth of road infrastructural development, particularly for low income economies where critical savings accruing from this mitigation effect could be invested in further development projects.

Laboratory simulation of flooding was carried out on stabilized soil test specimens. The production of the test specimens involved the use of Lower Oxford Clay (LOC) as a typical low-bearing capacity soil, stabilized using varying proportions of a latently hydraulic industrial by-product material – Ground Granulated Blastfurnace Slag (GGBS) – blended with quicklime (CaO) as an activator. Reducing the use of lime by incorporating GGBS enhances sustainability of the proposed process. Cylindrical test specimens of 50 mm in diameter and 100 mm in height were statically compacted to achieve maximum dry density (MDD), prior to moist curing for varying time periods before simulating prolonged flooding. Water absorption was monitored during the flooding process, and the specimens then tested for compressive strength. The results suggest potential of these lime–GGBS–LOC mixtures in overcoming the deleterious effect of flooding, by reduced water absorption and maintenance of a rigid cementing matrix structure after prolonged flooding. This is believed to offer savings from the improved robustness of road structural layers upon flooding, through resource conservation and material durability enhancement.

1. Introduction

Human activities have exerted enormous pressure on the global economy over the past few decades. This is due to the need to meet the vast demands of the world's ever increasing population. However, the securing of these needs is hugely tied to natural resources (fossil fuel, land, environment etc.). The large scale demand for these limited natural resources requires large scale production by different industries. This has the cumulative effect of depleting scarce natural resources and increasing environmental pollution, with by-products such as nitrous oxide (NO_x), carbon dioxide (CO_2), water vapor, ozone, methane and chlorofluorocarbons (CFC's) being liberated to the atmosphere. The cumulative effect of these activities has been evaluated to be responsible for the green house effect, which in turn causes global warming (Attom and Al-Sharif, 1998; Carpenter, 2004; Hegerl et al., 2007; Rao et al., 2008; Kilsby et al., 2009; Acquaye and Duffy, 2010; Lin, 2010; Norgate and Haque, 2010).

The effect of water vapor as one of the major contributors to the green house effect is aggravated by the increase in industrial operations over the years. This has the potential of increased frequency and severity of rainfall (Kilsby et al., 2009) leading to flooding events. Flooding has a negative impact on development as long staying flood waters could be deleterious to the structural layers of roads, material components of buildings (fired and unfired clay bricks, and rammed earth) and compromise their bearing strength. For design solutions involving low load bearing soils, soil stabilization has been employed with a view to improving the factor of safety.

The stabilization of soft and low strength soil with lime has a long history (Bell, 1996; Greave, 1996; Smith 1996; López-Lara and Castaño, 2000; Puppala et al., 2003; Barker et al., 2007). There is a growing tedency to replace the traditional stabilizers with either natural, industrial or other by-product materials for use in soil stabilization, either for cost considerations, environmental conservation or enhancement of material engineering properties. This line of thought has already been investigated by numerous investigators (Kamon and Nontananandh, 1991; Kinuthia et al., 1999; Wild et al., 1999; Al-Rawas et al., 2005; Higgins, 2005; James et al., 2008; Oti et al., 2009, to name a few). Oti et al. (2009) have recently reported on effort to reduce energy consumption through reduced use of fired building to a more

* Corresponding author. Tel.: +44 7765107540.
E-mail address: gobuzor@yahoo.com (G.N. Obuzor).

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sustainable approach of waste material utilization. GGBS has been established to possess the attributes of strength enhancement by engaging the free lime in the lime systems to form more C-S-H gel (Wild et al., 1999; Puppala et al., 2003; Oti et al., 2009). The gel further crystallizes to form the matrix structure that resists volumetric instability and enhanced strength. This reaction which is time dependent has also been reported to aid the densification of the cementing matrix structure (Puppala et al., 2003). It is believed that the use of alternative binders and assessment of the manner in which infrastructure development is achieved will enhance environmental management, with the global benefits of reduced carbon footprint, conservation of scarce natural resources, environmental regeneration and cost efficiency ensuing from the mitigation of the damage done to road structural layers by flooding.

Although there is a significant body of research on the subject of soil stabilization, there is little or no literature on the stabilization of low bearing capacity soils with a view to mitigating the effects of flooding. The aim of the current research reported here was therefore to explore the possibility of achieving significantly high degree of stabilization, so as to resist the effects of flooding, and to assess the robustness of stabilizing flood susceptible soils by partially replacing the amount of lime traditionally used for stabilization with GGBS, an industrial by-product material from the manufacture of steel. In order to achieve such a high degree of stabilization, the current research aimed to utilize a high stabilizer content of 16% to stabilize Lower Oxford Clay (LOC). This is against the traditional practice of using 3%–8% lime or 3%–5% for cement. It was hypothesized that a higher stabilizer would reduce the deleterious effects of flooding, but still be economical to use as opposed to facing significant road washout during flooding, with expensive reconstruction. Traditionally, California Bearing Ratio (CBR) test has been used for the assessment of strength of soil for road construction. Nevertheless, there have been trends adopted, as alternative approaches, to use unconfined compressive strength (UCS) test, for the evaluation of the strength of soil used for the purpose of road works (Thomas, 2002). In the current research, cylindrical test specimens measuring 50 mm in diameter and 100 mm in length were produced from samples of Lower Oxford Clay mixed with varying dosages of stabilizer at varying compaction moisture contents. Three blended mix compositions of stabilizer binders were adopted (4%Lime-12%GGBS, 8%Lime-8%GGBS and 16%Lime-0%GGBS). These blends were arrived at after progressively replacing lime with Ground Granulated Blastfurnace Slag (GGBS). A wide range of lime-GGBS formulations had been tried, and the ones reported here are those that showed best potential. Similarly, three compaction moisture contents were adopted (23%, 28% and 33%) after determining the OMC and MDD of the LOC by proctor compaction test in accordance with British Standard BS 1377. The cylindrical test specimens were produced after carefully mixing the dried LOC with the lime-GGBS blended stabilizers at the predetermined moisture contents. Static compaction pressure was applied using a hydraulic jack. A steel mold fitted with a collar was used to accommodate enough material to produce one specimen at a time, to be compressed to achieve the required dimensions. The samples were subjected to soaking in water for a prolonged period, to simulate a flooding event. This was carried out after the test specimens had been moist cured for different time periods. It has been observed from the results that the introduction of GGBS reduced water absorption upon soaking in water and also enhanced strength development of the soaked specimens.

2. Materials

2.1. Soil

Lower Oxford Clay, (LOC) was used as the target soil for the investigation. This stems from the hypothesis that if the expansive behavior of this clay normally observed upon stabilization with lime (Kinuthia et al., 1999; Wild et al., 1999) can be effectively managed, then, any other scenario presented by any other soil type is likely to be manageable to some reasonable degree. The material was therefore seen as providing an ideal test for the blended stabilizers used, and because of the material's propensity to expand, an ideal test for simulating severe effects of flooding. The disturbed LOC material was dried at room temperature and homogenized upon receipt from the supplier Hanson Brick Ltd., Stewartby, Bedford, UK. The mineralogical analysis carried out by Hanson Brick Ltd. shows the LOC to contain 23% illite, 10% kaolinite, 7% chlorite, 10% calcite, 29% quartz, 2% gypsum, 4% pyrite, 8% feldspar and 7% organic substances (Higgins et al., 2002). Table 1 shows the oxide and mineralogical composition of the LOC from Hanson Brick Ltd., and some chemical composition data, which include carbonate sulfate, sulfur (as sulfide), chloride and soluble silica.

2.2. Lime

The lime used was quicklime (calcium oxide), supplied by Buxton Lime Industries Ltd., in the form of a white powder. Its oxide composition and some of its physical properties are represented in Table 2. This was preferred to slaked lime due to its reactivity and faster moisture drying characteristics as well as the environmental implication of reduced dust level.

2.3. Ground Granulated Blastfurnace Slag (GGBS)

Ground Granulated Blastfurnace Slag (GGBS) was supplied by Civil and Marine Slag Cement Ltd., Llanwern, Newport, South Wales, UK. Table 2 shows its chemical composition and physical properties.

Table 1
Oxide, mineral and chemical composition analyses of the Lower Oxford Clay (LOC) (Wild et al., 1998; Higgins et al., 2002).

	Composition (wt.%)
Oxide analysis	
SiO_2	46.73
Al_2O_3	18.51
CaO	6.15
Fe_2O_3	6.21
MgO	1.13
K_2O	4.06
Na_2O_3	0.52
TiO_2	1.13
FeO	0.8
Mn_2O	0.07
P_2O_5	0.17
LOI	15.79
Mineral analysis	
Chlorite	7
Illite	23
Gypsum	2
Kaolinite	10
Quartz	29
k-feldspar	8
Calcite	10
Pyrite	4
Chemical and other analyses	
Soluble silica	0.43
Carbonate (CO_3)	5.02
Chloride (Cl)	0.01
Sulphide (S_2)	0.018
Sulphate (SO_4)	1.29
Total sulphur	1.5
Organics	7

Table 2
Oxide and chemical composition, and some physical properties of quicklime, Portland cement and GGBS.

	Composition (wt.%)	
	Quicklime	GGBS
Oxide analysis		
SiO_2	0.9	35.34
TiO_2	–	–
Al_2O_3	0.15	11.59
Fe_2O_3	0.07	0.35
MnO	–	0.45
$CaCO_3$	2.2	–
CaO	95.9	41.99
MgO	0.46	8.04
S_2	–	1.18
SO_3	–	0.32
Physical properties		
Specific gravity	2.3	2.9
Bulk density, kg/m^3	480	–
Colour	White	Off-white
Insoluble residue	–	0.3
Glass content	–	≈90

3. Methodology

Stabilized LOC using varying levels of lime – GGBS blends at different compaction moisture content levels were compacted into cylindrical moulds measuring 50 mm in diameter and 100 mm in length. The LOC which was carefully dried of moisture was mixed with the stabilizer with a hand mixing technique to enhance the homogeneity of the mix. For each cylinder, the wet materials were placed in a steel mould fitted with a collar so as to accommodate all the material. The material was then subjected to a static compression using a hydraulic jack to achieve the desired maximum dry density (MDD) at a constant volume. The cylinders were then extruded by the application of a steel plunger lubricated with a thin film of oil. The cylindrical specimens were then wrapped with cling film to ensure that further drying was minimized. This was further enhanced by placing the specimens in plastic containers throughout the period of curing. These plastic containers also helped in the regulation of the humidity at which the samples were cured before testing. At the end of each curing period (7, 14, 28, 56 and 90-days), two cylindrical specimens per mix composition were unwrapped of the cling film covering and completely soaked in water. The soaking was controlled by soaking the same mix composition in the same container to avoid cross contamination of samples. The samples were soaked for a period of 10-days and their water absorption rate monitored by weighing them hourly using a high sensitivity weighing balance build to a weighing error of ± 2 g. After the 24-hourly monitoring, another series of monitoring regime was initiated, lasting for a total period of 10-days wherein samples were weighed on a daily basis. This was followed by another testing protocol in which the samples were subjected to Unconfined Compressive Strength (UCS) test at a strain rate of 2 mm per minute, using a Hounsfield Compression Testing machine, capable of loading up to 10 kN. This was aimed at evaluating the soaked strength of the different samples.

4. Results

The water absorption investigation was very extensive, cutting across all the blended mix compositions of 4%Lime-12%GGBS; 8% Lime-8%GGBS and 16%Lime-0%GGBS at the various compaction moisture contents of 23%, 28% and 33%. The curing and testing regimes followed the 7, 14, 28, 56 and 90 days established earlier. The graphs are grouped based on moisture contents or curing ages. Fig. 1 is a representation of the water absorption profile of stabilized

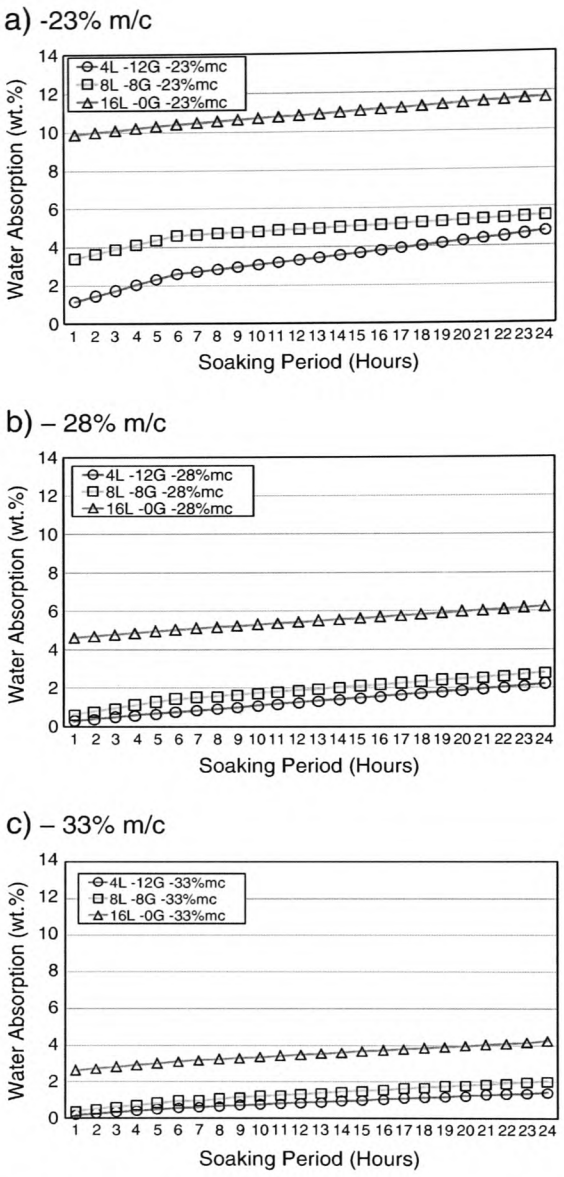


Fig. 1. Water absorption profile of stabilized Lower Oxford Clay after 7-days of moist curing and 24 h of complete soaking in water.

Lower Oxford Clay (LOC) compacted at varying compaction moisture content, after 7-days of moist curing and 24 h of complete submersion in water. The samples in which there was no replacement of lime (16% lime-0% GGBS) at compaction moisture content of 23% (Fig. 1a) showed a high affinity for water, absorbing up to 10 wt.% of water after 1 h of complete soaking in water. The rate of water absorption increased uniformly to about 12 wt.% after 24 h, representing an increase of about 20% from the time of initial absorption to the lapse of 24 h. Relative to the lime replaced samples, this represents a huge absorption, expansion and volume instability. The 8% Lime-8%GGBS and 4%Lime-12%GGBS stabilized at 23% moisture content (Fig. 1a) showed minimal absorption of above 3% and above 1% respectively, with an overall amount of water absorbed not exceeding 6% for the 8%Lime-8%Lime-8%GGBS and 5% for the 4%Lime-12%GGBS blended compositions. However, the general trend observed for all the samples is one of increasing amount of water absorption with extended time of submersion and an increase in the rate of absorption with submersion time to a certain value where the rate of water absorption

smoothened to form an almost flat graph. Secondly, the water absorption reduced upon the reduction in the amount of lime in the system. From Fig. 1b the trend is the same as that in Fig. 1a. However, the rate and amount of water absorbed are obviously less, reducing from 10% for the 16%Lime-0%GGBS at 23% compaction moisture content, to about 5% for 16%Lime-0%GGBS at 28% compaction moisture content. The overall water absorption for the 16%Lime-0%GGBS stabilized at 28% compaction moisture content is about half of the absorption for the same blend at 23% compaction moisture content after a curing period of 7 days. There is still a further reduction in water absorption with increase in the compaction moisture content as shown in Fig. 1c. This crystallizes yet another trend, increasing the compaction moisture content of the different blended mixes after 7 days of moist curing and 24 h of soaking reduces the amount of water absorption. Figs. 1a, b and c are to guide us towards understanding what is happening within the early period of complete submersion of samples in water. However, Figs. 2 to 6 picture what happens between the first-day to the 10th day of subjecting these samples to water by complete immersion after curing for varying ages of moist curing before testing.

Fig. 2a, b and c are the water absorption profiles of stabilized Lower Oxford Clay compacted at 23%, 28% and 33% compaction

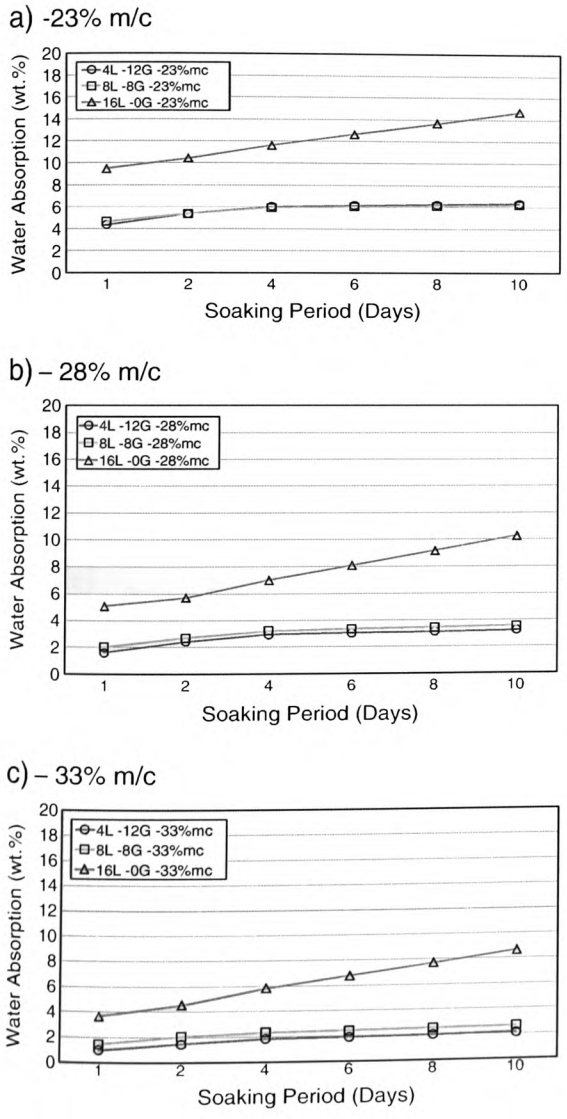


Fig. 2. Water absorption profile of stabilized Lower Oxford Clay after 7-days of moist curing and 10 days of complete soaking in water.

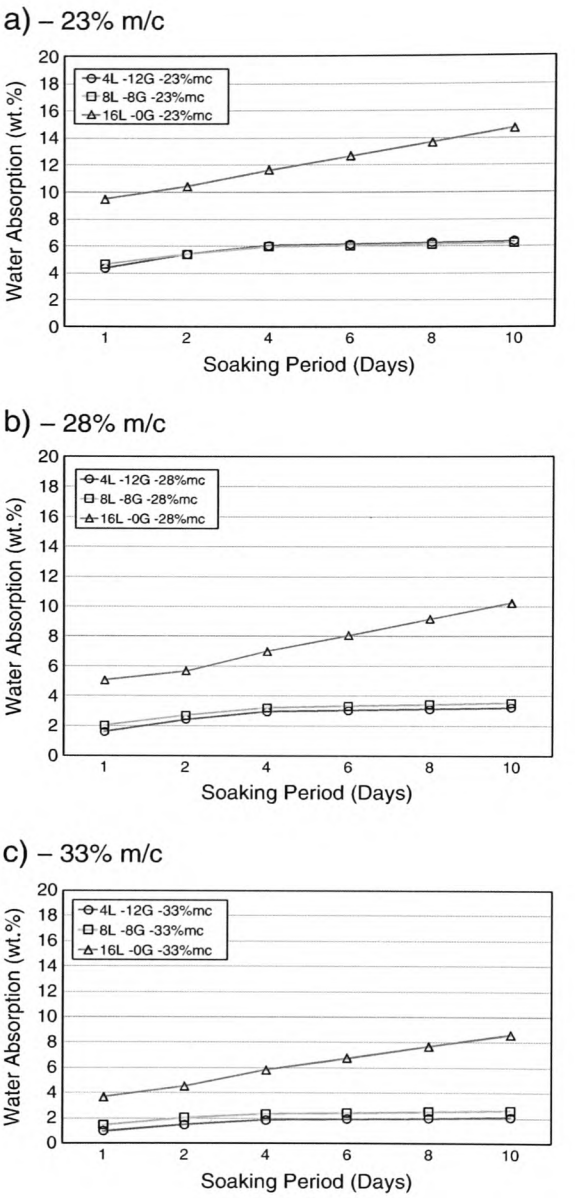


Fig. 3. Water absorption profile of stabilized Lower Oxford Clay after 14-days of moist curing and 10 days of complete soaking in water.

moisture contents after 7-days moist curing and 10-days complete soaking in water. The trends are the same when compared with those of 24-hour soaking. With the 16% lime-0% GGBS at the different moisture content levels of 23%, 28% and 33% Fig. 2a, b and c show high rate of moisture demand which reduced with increase in compaction moisture content. Secondly, replacing lime with GGBS by 50% and then 75% as shown by the 8%Lime-8%GGBS and 4%Lime-12%GGBS stabilizer blends, played a significant role in the reduction of water absorption. The 8%Lime-8%GGBS and 4%Lime-12%GGBS stabilized at the various compaction moisture contents showed a comparative reduction in water absorption. This has the potential of enormous savings accruing from the mitigation of the distress occasioned by water absorption, expansion and volume instability, experienced on road sub-grade and sub-base of lime stabilized road structural layers. Similarly, strength reduction is improved owing to the impermeable nature of these stabilized materials. There is also a sustained reduction in water absorption as a consequence of reduction in the amount of

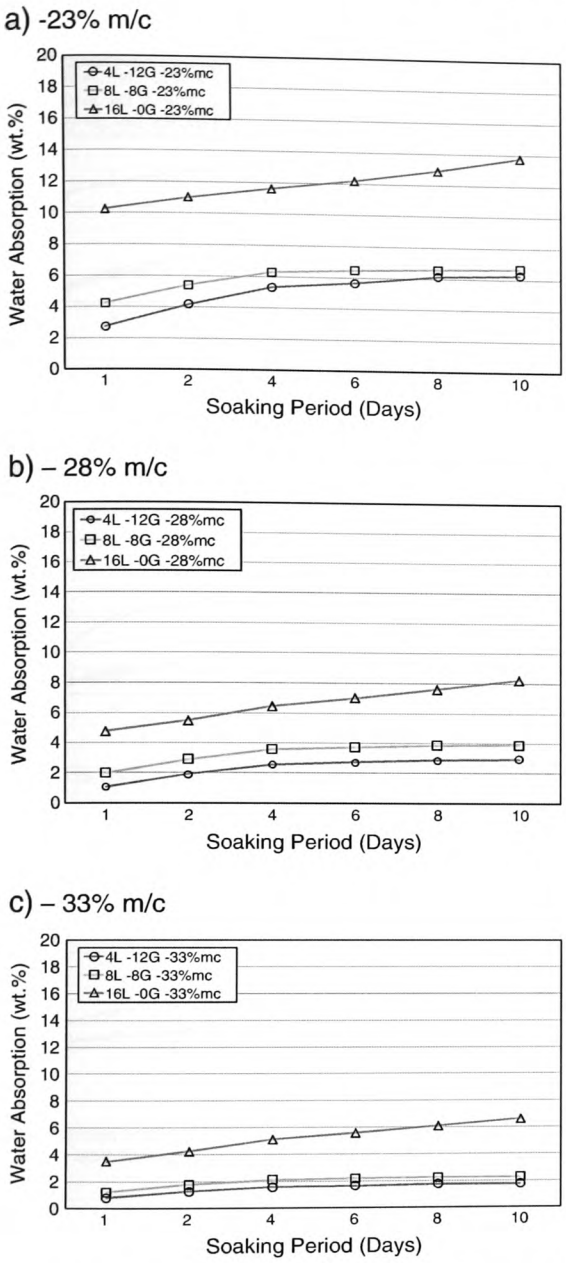


Fig. 4. Water absorption profile of stabilized Lower Oxford Clay after 28-days of moist curing and 10 days of complete soaking in water.

mixing lime content which also translates to cost savings. The water absorption is observed to reduce progressively with curing age till 56 days moist curing period. Further reduction in water absorption is still evident from (Figs. 3, 4, 5 and 6) which are the profiles of the14-, 28-, 56- and 90-days moist curing and submersion periods respectively. This further reinforces the fact that age has a great influence on the water absorption characteristics of these novel material formulations. Fig. 6a, b and c are the water absorption representations of stabilized LOC-lime-GGBS system compacted at 23%, 28% and 33% compaction moisture content after 90-days of moist curing and 10-days of complete soaking in water. The same trend is observed in the three figures, but in this case there is another behavior displaced both by the lime and the lime-GGBS system. The water absorption at this age is observed to be more than that at 56-days which following the trend from the early ages could be termed anomalous.

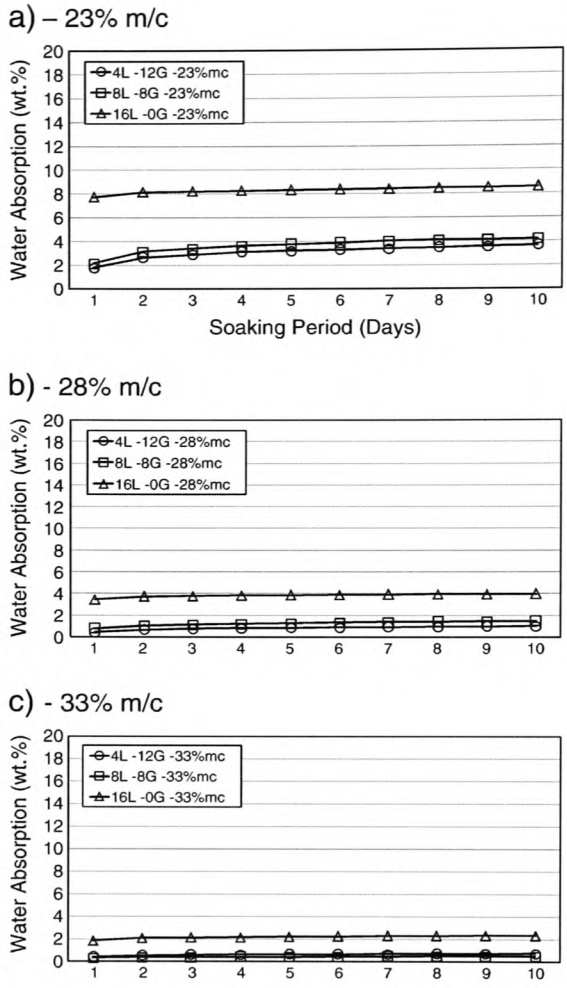
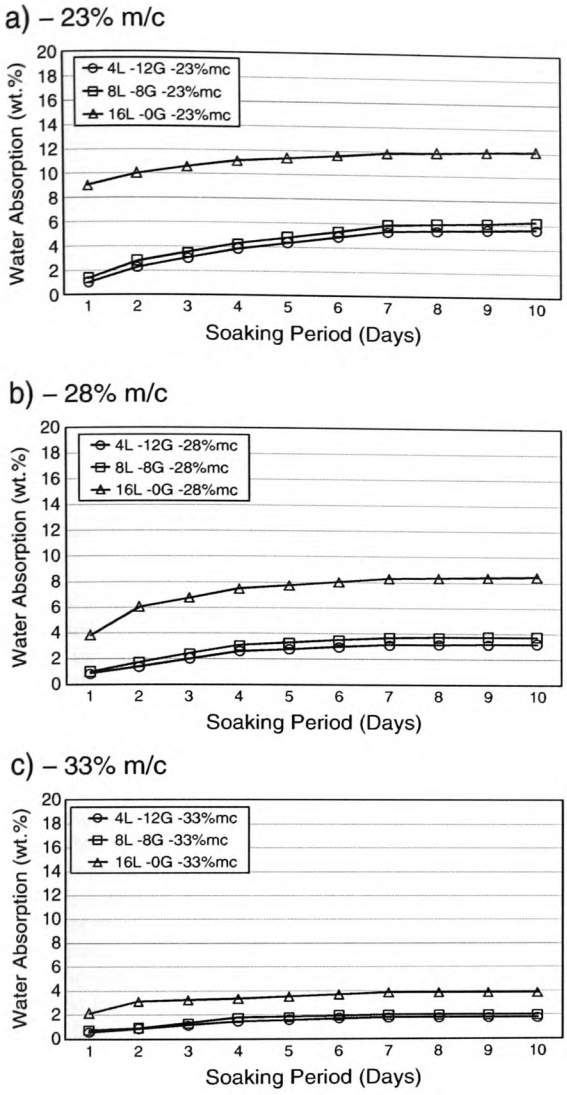


Fig. 5. Water absorption profile of 28% compaction moisture content level stabilized Lower Oxford Clay after 56-days of moist curing and 10 days of complete soaking in water.

However, the strength reduction index test also confirms this through higher strength values at 56-days than at 90-days. This anomalous behavior is under investigation to check the permeability, parameter of the materials. The findings of which will be reported in a future publication.

Fig. 7a, b and c show the soaked Unconfined Compressive Strength test (UCS) values of stabilized LOC-Lime-GGBS cylinders compacted at 23%, 28% and 33% moisture content, and subsequently tested after different ages of moist curing (7, 14, 28, 56 and 90-days). From Fig. 1a the 16%Lime-0%GGBS-23% moisture content showed very low strength gain. Replacing lime with GGBS at 50% and 75% as illustrated by 8%Lime-8%GGBS and 4%Lime-12%GGBS produced a significant effect on the soaked strength. The 4%Lime-12%GGBS and 8% Lime-8%GGBS at 23% compaction moisture content showed higher strength values, from strength values of 1250 kPa and 1000 kPa at 7-days to 2000 kPa and 1600 kPa upon submersion in water for 10-days after a moist curing period of 56-days respectively. Between the 56-days and 90-days the soaked strength dropped to values slightly above 1500 kPa. This was unexpected, but the reasons for the possible causes of this trend are discussed later, with reference to the trends shown in Fig. 8 which shows the variations in water absorption for test specimens soaked after various periods of moist curing. There is an increase in soaked strength with curing age and with reduction in the percentage of mixing lime. More absorption of water



ig. 6. Water absorption profile of 33% compaction moisture content level stabilized ower Oxford Clay after 90-days of moist curing and 10 days of complete soaking in ater.

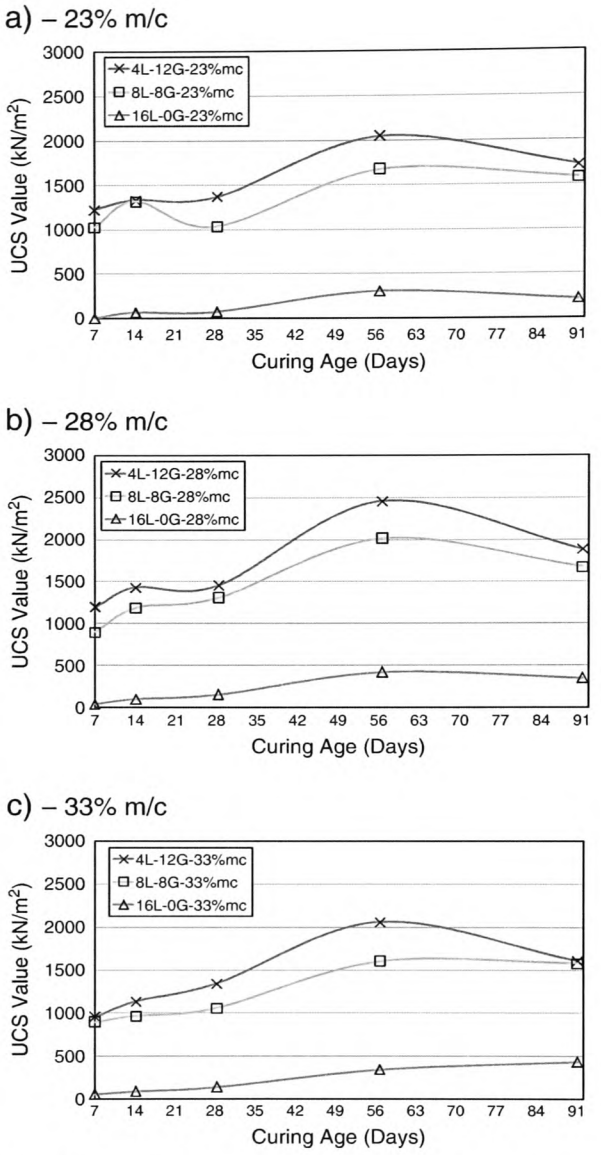


Fig. 7. Unconfined compressive strength of stabilized Lower Oxford Clay after different curing periods and upon 10-days of complete submersion in water.

epresents lower strength values. At 28% compaction moisture content (Fig. 7b) higher strength values were observed especially after 56-days curing, with the soaked strength still increasing with curing age and reduction in lime addition. A comparison of the soaked and unsoaked Unconfined Compressive Strengths have already been made where it was reported that the samples without lime replacement lost more strength upon soaking in water compared to lime replaced samples. It is also true that the more water absorbed by the system the lower the strength that is attainable.

5. Discussion

The compaction of soil is always targeted at its optimum moisture content (OMC) so as to achieve maximum dry density (MDD). When this density is achieved, it is evident that the best compaction effort has been reached with the benefit of high bearing capacity (Prusinski and Bhattacharja, 1999). However, in practice, to actualize the best performance, soil is always better compacted at controlled moisture contents, marginally higher than the OMC as this accommodates any moisture losses during the compaction operation, and is also more likely to result in less brittle failure in service. Moreover, when lime is involved in the stabilisation process, the water demand is usually higher for sustaining the pozzolanic reactions involving lime and the clay minerals, which is very exothermic in nature (Bell, 1996; Prusinski and Bhattacharja, 1999). Similarly, when lime blended with GGBS is added to clay soil, the pH of the system is changed to favor the reaction between calcium, silicate and aluminate ions comprising the clay to form calcium silicate hydrate (CSH) and calcium aluminate silicate hydrate (CASH) gels that form the matrix structure, for strength development of stabilized systems upon crystallization (Bell, 1996; Tasong et al., 1999; Wild et al., 1999; Puppala et al., 2003; Barker et al., 2007 and Hafez et al., 2008). From (Figs. 1–6), it was noted that higher water absorption was associated with lower compaction moisture contents. However, at increased compaction moisture contents the water absorption was reduced. This could be interpreted as, higher compaction moisture contents provided lower inter-particle friction to aid compaction and optimized particle packing arrangement. Subsequently, the void ratio was reduced in comparison with that of those test specimens

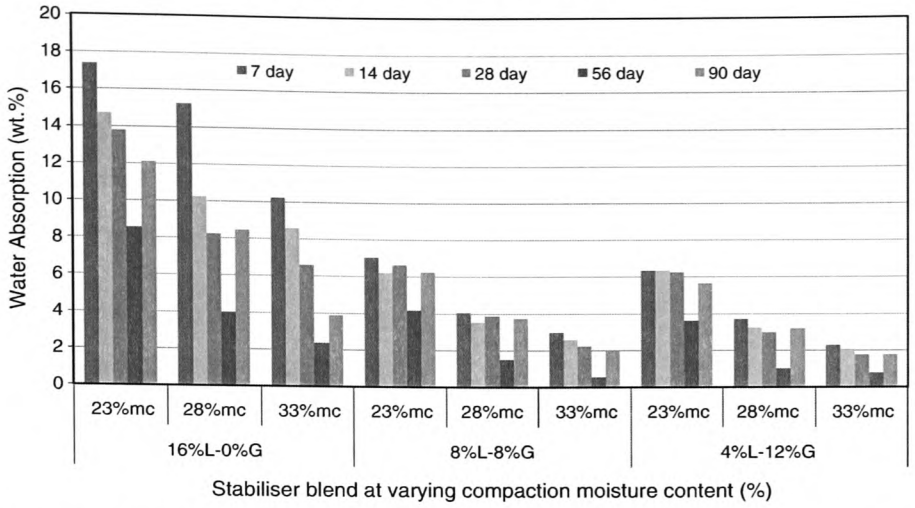


Fig. 8. A comparison of water absorption within the different stabilizer binder mix compositions at varying compaction moisture contents at the end of the 10 days soaking period.

compacted at lower moisture contents. Also, the more lime in the system resulted in more water absorption. According to Bell (1996), the hydroxyl ions present in a lime-stabilized clay system modify the affinity of the surfaces of the clay particles for water. Hence, the addition of lime to clay materials increases their optimum moisture content. The author further suggested that the amount of water available for hydration reactions to form cementitious bonds influences the strength which can be achieved. This implies that a low compaction water content of a mix means that not only is there insufficient water for efficient compaction but also that the little water available is rapidly used in the hydration process such that the peak strength is achieved after a short period of time only. Therefore, in order to develop maximum strength it is necessary to use a mix water content slightly in excess of optimum moisture content.

The importance of age in the soil stabilization process could also be deduced from the graphs. At lower curing ages the water absorption was higher but reduced with extended age of moist curing. The strength of a lime-GGBS stabilized system is dependent on the complex hydration and the time dependent pozzolanic reactions involving clay minerals, lime and lime activated slag. This has a pore-blocking effect resulting in increased long-term hardening of the cement paste. The blocking of pores leads to higher strength and lower permeability (Wild et al., 1998; Tasong et al., 1999). On this premise it could be argued that, at extended age of curing, the system densification was enhanced and water absorption was consequently reduced. This also explains why the strength of samples showed a general increase with curing age even upon soaking for a ten day period. The soaked strength values increase with increase in the compaction moisture content to a point of highest strength at 28% compaction moisture content and then reduced with further addition of water. However, when compared with the results of the water absorption, better (lower) absorption values were achieved at higher compaction moisture contents. This is believed to be from the better compaction due to reduced inter-particle friction thereby increasing particle packing and particle bonding. Further research is ongoing to validate claims that increased hydration products from the hydration of GGBS was responsible for increasing the density and blocking water passage. This will involve investigation of the permeability of the stabilized cured material. The reduction in strength when test specimens were soaked after 56 days of moist curing is also being investigated further. The authors hypothesize that from the results shown in Fig. 8, specimens that were soaked at a curing stage when pozzolanic reactions were still ongoing (curing up to 56 days), benefited from soaking. In contrast, specimens cured at beyond this period, irrespective of the compaction moisture content, had reached peak development and any subsequent soaking could only lower the strength. This has significant practical implications, as the material that would be expected to withstand soaking is the one showing reduction in strength upon soaking. This is being thought through further, and will be re-addressed in subsequent publications by the authors.

6. Conclusions

The present study has demonstrated that it is possible to enhance the compressive strength and reduce the water absorption of low load-bearing capacity soils to resist the effects of flooding, by the addition of lime blended with GGBS. The following conclusive remarks can be drawn from the investigation.

1. Test specimens with lower compaction moisture content absorbed more water upon complete soaking in water compared to specimens compacted at a relatively higher moisture content. This was partly because compaction at the lower moisture content increased inter-particle friction and partly due to the incomplete formation of hydration products compared with the situation prevailing at the higher compaction moisture content — less inter-particle friction, coupled with more complete hydration and blockage of voids.
2. Increase in lime addition resulted in increased water absorption of the LOC-lime-GGBS stabilized system. High lime contents increased the amount of free lime in the system which required more moisture for hydration. Coupled with the increased production of heat owing to the exothermic nature of the lime–water reaction, there was increase in water absorption to sustain the demand for water.
3. The water absorption was reduced with the addition of GGBS and increasing the age of curing. GGBS enhances the formation of the gels responsible for strength development (primarily C-S-H gel), in a time dependent process. This results in the densification of the matrix structure of the system reducing the micro-pores that accommodate water.
4. Higher water absorption capacity was observed after 90-days of moist curing compared to the trend at 56-days. It was also observed that the 56-days soaked strength values were higher than those at 90-days. This situation results from the stage of hydration at the time of soaking in water, whereby specimens where

hydration is complete resulted in reduced strength. Further investigations are on-going to elucidate this complex situation.

knowledgments

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Technical Note

Utilisation of lime activated GGBS to reduce the deleterious effect of flooding on stabilised road structural materials: A laboratory simulation

G.N. Obuzor *, J.M. Kinuthia, R.B. Robinson

Civil Engineering Research Unit, Department of Engineering, Faculty of Advanced Technology, University of Glamorgan, Pontypridd, CF37 1DL, UK

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ABSTRACT

Reducing the high embankments associated with road construction especially in low-land areas to within the design flood level, will offer project overall cost reduction amidst enhancing environmental friendliness. This is necessary to mitigate the global environmental concerns of flooding envisaged to be more critical in low-land areas and to keep up with the pressure exacted on land due to road infrastructural development projects, it is important therefore to develop more sustainable technologies. The utilisation of marginal, waste and/or by-product materials such as Ground Granulated Blastfurnace Slag (GGBS) to modify the engineering properties of locally available geomaterials such as local soils could be handy. Laboratory tests such as Unconfined Compressive and linear expansion tests were carried out to simulate the effects of flooding on the road layers in terms of strength and durability, against existing design criteria for submerged materials. Cylindrical test specimens of 50 mm in diameter and 100 mm in length were produced from Lower Oxford Clay stabilised with lime–GGBS stabiliser materials. Linear expansion tests were conducted and an alternative durability testing regime of soaking in water was also followed. The results obtained demonstrate that with careful selection and research, the local materials are capable of achieving structurally sound and environmentally friendly properties for durability comparable to, and sometimes better than, materials formulated using the more expensive and increasingly unsustainable traditional stabilisers such as Portland Cement (PC) or even lime. These suggest viability and feasibility of enormous savings as more marginal materials are used to replace virgin and classical material.

1. Introduction

Mitigating the effects of flooding on road embankments will be a boost to global economic development particularly for low income countries, where the huge savings from reduced embankment heights to within the flood lifeline and design flood levels in flooded areas could be invested in further infrastructural development and maintenance of existing infrastructures. Flooding has been a frequently occurring natural disaster, however, with the contemporary environmental issues of global warming more flooding is anticipated.

Low-land areas are the most susceptible to flood inundation. After cycles of flooding and drying, road embankments are often left with cracks. As has been pointed out by (Ampadu, 2007), these cracks are sources of water ingress to road sub grade, sub-base and foundations of other civil engineering facilities. This presents the possible danger of reduced bearing strength of soils, to levels below the allowable minimum for support of any facilities constructed on them and hence results in imminent failures.

Cracks can compromise performance if they become wide and admit significant moisture (Atom and Al-Sharif, 1998). As reported by (Basha et al., 2005), several techniques have been employed to minimise this problem, including compaction at moisture contents slightly drier than optimum; pre-cracking through inducement of weakened planes or early load application; delayed placement of surface hot mixes; reduced cement content in cement stabilisation; and use of interlayers to absorb crack energy and prevent further propagation. On the other hand, Greaves (1996) attributed the loss of strength of local soils upon soaking to the unsaturated state of such soils.

Soil stabilisation is the handiest option that could yield a positive contribution in terms of economic and resource sustainability since it allows enhancement of both standard and substandard in-situ soils to levels consistent with the requirements of a given application and thereby bringing in important engineering properties like improved strength; improved resistance to fracture, fatigue, and permanent deformation; improved resilient properties; reduced swelling; and resistance to the damaging effects of moisture (Greaves, 1996; Higgins et al., 2002; Basha et al., 2005).

The main objective of the work presented in this paper is to utilise locally available geo-, natural, and industrial waste and by-product materials, for the construction of sustainable road embankments and

* Corresponding author. Tel.: +447765107540.
E-mail address: gobuzor@yahoo.com (G.N. Obuzor).

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road structural layers, in order to reduce cost of construction as well as conserve the environment. This is with a view to preventing damages done to road embankments by flooding as a result of soil strength losses. The mitigation action will be achieved by increasing the bearing capacity of the soil. This will demonstrate that industrial waste and/or by-product materials such as GGBS can be used in a beneficial manner without any adverse impact on the environment.

This laboratory process was carried out on Lower Oxford Clay to demonstrate that its expansive characteristics upon stabilisation with lime in the presence of sulphates are avoidable. From the Unconfined Compressive Strength of both the soaked and unsoaked specimens and the volume stability test carried out using an automatic monitoring Linear Expansion device, there is the possibility of reducing the overall cost of construction amidst better environmental management option.

2. Materials

2.1. Soil

Lower Oxford Clay, (LOC) was used as the target soil for the investigation. This was based on the assumption that if the well-established expansive behaviour of this clay normally observed upon stabilisation with lime can be effectively managed, then any other scenario presented by any other soil type is likely to be manageable to some reasonable degree. The LOC which was dried at room temperature and homogenised was supplied by Hanson Brick Ltd., Stewartby, Bedford, UK. The mineralogical analysis carried out by Hanson Brick Ltd shows the LOC to contain 23% illite, 10% kaolinite, 7% chlorite, 10% calcite, 29% quartz, 2% gypsum, 4% pyrite, 8% feldspar and 7% organic substances (Higgins et al., 2002). Table 1 shows the oxide and mineralogical compositions of the LOC, while Table 2 shows chemical composition and other data which includes carbonate, sulphate, sulphur (as sulphide), chloride and soluble silica.

2.2. Lime

The lime used was quicklime (CaO), supplied by Buxton Lime Industries Ltd., in the form of a white powder. Table 3 shows its oxide composition and some of its physical properties.

2.3. Ground Granulated Blastfurnace Slag (GGBS)

Ground Granulated Blastfurnace Slag (GGBS) was supplied by Civil and Marine Slag Cement Ltd, Llanwern, Newport, South Wales, United Kingdom. Table 4 shows its chemical composition and physical properties.

Table 1
Oxide and mineral analyses of the lower oxford clay (Higgins et al., 2002).

Oxide	Wt.%	Mineralogy	
SiO_2	46.73	Chlorite	7
TiO_2	1.13	Illite	23
Al_2O_3	18.51	Gypsum	2
Fe_2O_3	6.21	Kaolinite	10
FeO	0.80	Quartz	29
Mn_2O	0.07	K-feldspar	8
CaO	6.15	Plagioclase	–
MgO	1.13	Calcite	10
K_2O	4.06	Siderite	–
P_2O_5	0.17	Anatase	–
Na_2O_3	0.52	Pyrite	4
LOI	15.79	Apatite	–
		Organics	7

Table 2
Chemical composition of LOC (Higgins et al., 2002).

Chemical composition	(%)
CO_3	5.02
CaO	5.60
Cl	0.01
IR	91.6
S	0.018
SO_4	1.29
SiO_2	0.43
Total sulphur	1.50

2.4. Methodology

The methodology involved the stabilisation of LOC using varying levels of lime–GGBS blends. For each mix regime, forty compacted cylindrical test specimens measuring 50 mm in diameter and 100 mm in length were produced from dried LOC mixed with the stabiliser at pre-determined moisture contents of 23%, 28% 33% and 38% enveloping the OMC of the material, for both Unconfined Compressive Strength (UCS) and linear expansion tests. To promote mix homogeneity a hand mixing technique was employed. For each cylinder, the wet materials were placed in a steel mould fitted with a collar so as to accommodate all the material. The material was then subjected to a static compression using a hydraulic jack to achieve the desired dry density closest to the MDD with the volume kept constant. The cylinders were then extruded by the application of a steel plunger lubricated with a thin film of oil. The cylindrical specimens were then wrapped with cling film to regulate moisture movement. This was further improved by placing the specimens in plastic containers for the period of curing. These plastic containers also helped in the regulation of the humidity at which the samples were cured before testing.

At the end of each curing period (7, 14, 28, 56 and 90 days), two cylindrical specimens per mix composition at each of the different moisture contents were subjected to unconfined compressive strength (UCS) test at a strain rate of 2 mm per minute, using a Hounsfield Compression Testing machine, capable of loading up to 10 kN. In addition, another two specimens from each of the stabiliser blends and compaction moisture contents were soaked in water in labelled containers with the cling film wrappers on the samples removed. The linear expansion test was carried out in a Perspex cell, housing transducers that relayed expansion signals twice in every 24 h for a total period of 50 days. For the first seven days, the test specimens were moist curing before the subsequent addition of water to 10 mm above base of sample. The expansion signals picked up by the transducers were then transmitted via the different channels and stored in an MPX 3000 data logger. The data could be accessed for plotting and further analysis and reporting, by means of WINHOST software, prepared by VJ Tech Ltd., Berkshire, UK, the manufacturers of the MPX 3000 data logger.

Table 3
Oxide composition and physical properties of quicklime.

Oxide composition	(%)
CaO	95.9
SiO_2	0.9
Al_2O_3	0.15
MgO	0.46
Fe_2O_3	0.07
$CaCO_3$	2.2
Specific gravity	2.3
Bulk density, kg/m^3	480
Colour	White

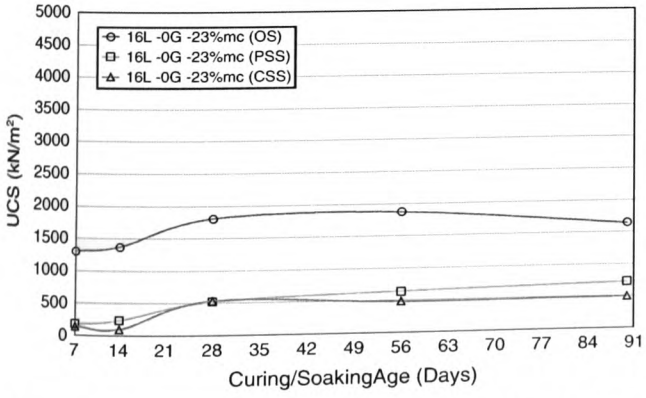
Table 4
Chemical composition and physical properties of GGBS (Wild et al., 1999).

Oxide	Composition (%) GGBS
CaO	41.99
SiO_2	35.34
Al_2O_3	11.59
MgO	8.04
Fe_2O_3	0.35
MnO	0.45
S_2	1.18
SO_3	0. 32
Insoluble residue	0.3
Relative density	2.9
Bulk density (kg/m^3)	1200
Colour	Off-white
Glass content	≈90

. Results

1. Unconfined compressive strength

These tests were carried out on four moisture content levels, this aper only presents those of 23% moisture level. Fig. 1 shows the UCS :st result of LOC stabilised at 16% binder addition. This stabiliser mix :gime (16%Lime–0%GGBS) where Lime was not replaced by GGBS owed very low strength development and rate of strength gain ith time. In the same way, submerging the samples in water, either artially or completely showed significant reduction in strength. owever, on replacing the lime content of the binder with 8% of GGBS attain a 1:1 ratio of replacement (8%Lime–8%GGBS) as shown in g. 2, the effect of soaking on stabilised LOC at a moisture content of 3% was significantly mitigated. There was the trend of increasing rength even under soaked environment. On further substitution of me with GGBS the 4%Lime–12%GGBS was reached which is a 1:3 tio of lime to GGBS (Fig. 3). The trend of increasing strength for the riginal (control) Samples, OS (not soaked), the Completely Soaked amples, CSS, and the Partially Soaked Samples (PSS) as designated the graphs was identified. Though the PSS showed little or no ain in strength between 14 and 28 days, it subsequently recovered achieve a UCS value of above 4000 kN/m^2 at 56 days with a gh rate of increase, then the rate slowed down between the 56 nd 90 days with an overall strength value of above 4500 kN/m^2. owever, for the PSS designated samples, the trend is that of a slow te of strength development from the 7 days testing to the 56 days here the samples suddenly picked up at a faster rate to achieve an verall strength that is a little less than 4500 kN/m^2. Similarly, the rate strength gain by the CSS was uniform and increasing at a low pace r the five levels of curing, soaking and testing.



1. Loss of strength upon soaking of LOC, stabilised with 16Lime–0GGBS at 23%mc.

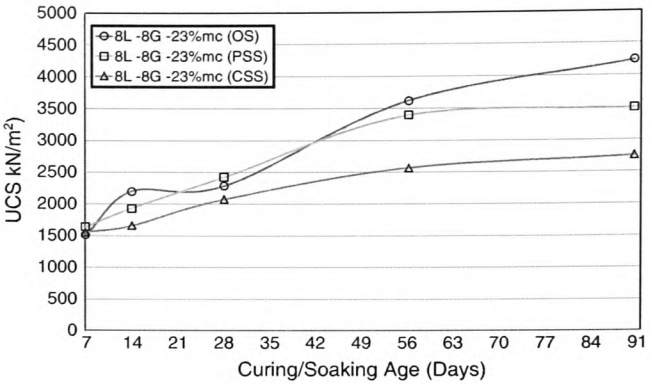


Fig. 2. Loss of strength upon soaking LOC, stabilised with 8Lime–8GGBS at 23%mc.

3.2. Linear expansion

Further investigations carried out on the durability of the novel material mixtures (LOC–Lime–GGBS) showed impressive results in terms of its Volume Stability. Figs. 4– 6 show the effects of the variation of the stabiliser blends (at four compaction moisture contents), on the linear expansion upon moist curing and subsequent soaking. The 16%Lime–0%GGBS blend as shown in Fig. 4, denotes high expansion from the moist curing period through the soaking period with the rate of increase slowing down with age. Another trend that is clearly established is the increase in expansion with the reduction in specimen compaction moisture content level and lime addition. Upon replacement of lime with GGBS to attain a blended composition of 8%Lime–8%GGBS (Fig. 5) the Volume Stability of the material was greatly enhanced with little expansion within the moist curing period. Further replacement of lime as shown in Fig. 6 (4%Lime–12% GGBS) followed the same trend as that in Fig. 5; however, the rate of Volume instability was further improved.

4. Discussion

From Fig. 3, all the test specimens stabilised with the 4%Lime–12% GGBS blend at 23% moisture content showed increase in their strength gain for all the samples. The OS samples started at a 7 days UCS value of approximately 2000 kN/m^2 and increased sharply to a value slightly above 2400 kN/m^2, the rate of increase slowed down, then increased again to 4000 kN/m^2 then to above 4500 kN/m^2 in 90 days. When lime and clay are mixed in the presence of moisture two things happen, the first being the modification reaction that makes the stabilised material workable, followed immediately by pozzolanic reaction driven by the pozzolanic nature of clay minerals. These

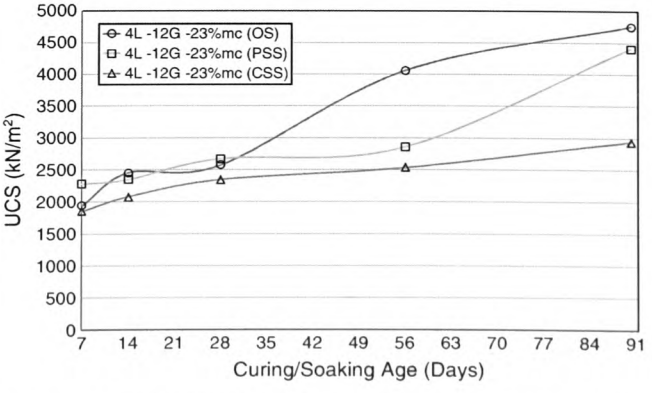


Fig. 3. Loss of strength upon soaking LOC, stabilised with 4Lime–12GGBS at 23%mc.

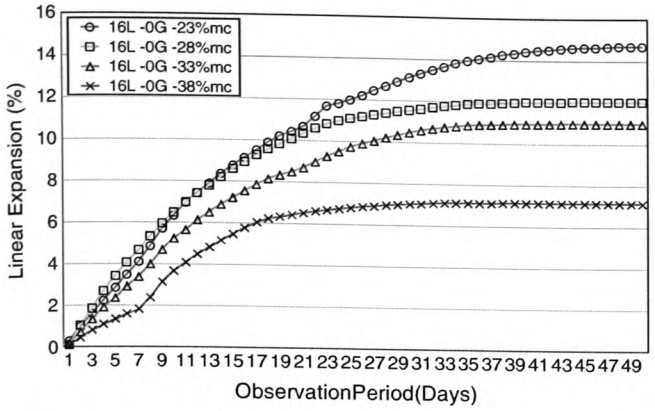


Fig. 4. Linear expansion of LOC stabilised with 16L–0G, upon moist curing for 7 days, followed by prolonged soaking in water.

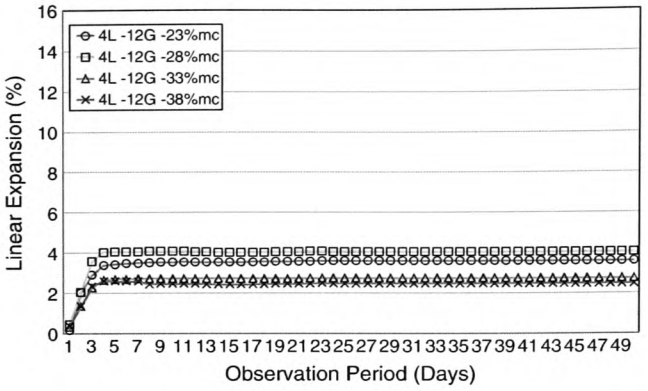


Fig. 6. Linear expansion of LOC stabilised with 4L–12G, upon moist curing for 7 days, followed by prolonged soaking in water.

reactions sometimes last for years giving the system that desirable quality of autogenic healing. When clay is mixed with lime in the presence of moisture there is usually a rapid ion exchange reaction, trailed by a slow chemical process which produces cementitious Calcium-Aluminate-Silicate-Hydrate gel (C-A-S-H) and in some cases crystalline calcium aluminate hydrate and calcium silicate hydrate phases. This produces the cementing effect that binds clay particles together to aid in resisting volumetric instability. The early and later strengths of stabilised mixtures are usually as a result of the complex hydration reactions that yield products which crystallise into the cementing matrix of any cementitious system (Holt and Freer-Hewish, 1996; Rogers and Glendinning, 1996; Wild et al. 1996; Prusinski and Bhattacharja, 1999).

Strength Reduction Index analysis carried out on the OS and CSS samples results showed a loss in strength, this could possibly be interpreted as: the Lime–GGBS–LOC system not having gained enough permeability to muster resistance to the ingress of water molecules. Hydration reaction yields CSH gel which increase permeability, density and strength according to Puppala et al. (2007) and Wild et al. (1999). During pozzolanic reactions, the free lime in the system goes into further reaction with the clay minerals to produce more CSH gel which helps in sealing porous pores and thereby enhancing strength. Though, the system needed moisture to carry on and complete the hydration reaction within the early ages but not as much as is possible by soaking them in water. It is anticipated that if samples are allowed to cure further more beyond 28 days before immersing in water, they should be able to present more resistance to water ingress translating to higher bearing capacities, this agrees with the conclusion of Wild et al. (1999) and the results obtained at 56 and 90 days. As reported by Greaves (1996) clay minerals are natural pozzolanas and possess the ability to react with lime added to the soil to produce cementitious products. The added lime results in an increased pH to a value in excess of 12 with a resultant increase in the solubility of siliceous and aluminous compounds which react with calcium to form calcium silicate hydrates and alumina hydrates. The CSH/CAH occurs initially in gel form to coat the soil particles, to form a bond, which eventually crystallises into calcium silicate/aluminate hydrates. The cementitious products are broadly similar in composition to those of cement paste. The process is relatively slow because the available lime has to diffuse through the soil structure and the initially cementitious products to the reaction sites. This reaction results in a gain in strength as reported by Prusinski and Bhattacharja (1999).

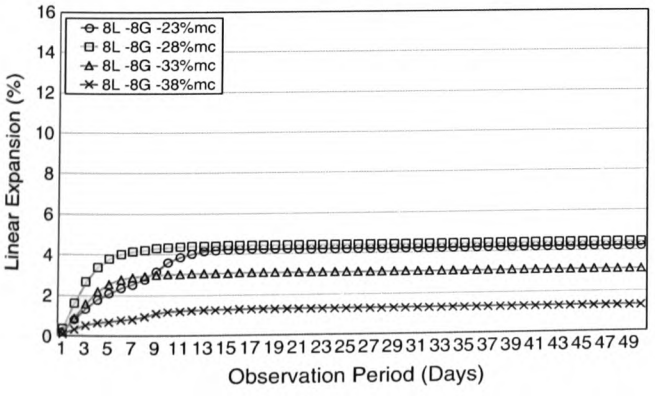


Fig. 5. Linear expansion of LOC stabilised with 8L–8G, upon moist curing for 7 days, followed by prolonged soaking in.

5. Conclusions

The following conclusions can be drawn:

1. That the presence of GGBS increased the formation of the CSH gel matrix which forms the cementing medium in soil stabilisation.
2. That at longer curing periods the GGBS was able to enhance the densification of the system to reduce water ingress as the behaviour of an embankment is hugely influenced by drainage characteristic of the fill material, leaving room for choice between a well drained material and one that is highly impermeable.
3. That the strength development of the cementitious system is a function of curing age and curing conditions (temperature).
4. That the expansion characteristics of the LOC–Lime–GGBS stabilised system is also a function of the percentage of lime in the system. And the expansion generally occurred within the first 72 h of the samples production.
5. That the amount of compaction moisture content of the stabilised system determined the expansion properties, where samples compacted at higher moisture contents showed lower expansion to those with lower moisture content. This is believed to be due to better compaction with higher moisture content and less void spaces resulting from reduced particle to particle friction thereby aiding particle packing arrangement and compressibility.

Acknowledgements

The authors will like to acknowledge the support of Total E & P Nigeria Limited (EPNL) for their partial sponsorship of this project and their commitment in human resource development initiatives within their operational areas. Also the ceaseless efforts of the Division of Civil and Mechanical Engineering, and the technical staff especially Darren Crocker of the soils lab for his tremendous assistance.

The authors would also like to acknowledge the incredible support from Buxton Lime Industries Ltd, Hanson Brick Ltd., Civil and Marine Slag Cement Ltd for the supply of lime, Lower Oxford Clay, and Ground Granulated Blastfurnace Slag respectively.

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Sustainable soil stabilisation for road embankments in flood plains

Kinuthia J. M., Obuzor, G. N and Robinson R.

Civil Engineering Research Unit, Department of Engineering, Faculty of Advanced Technology, University of Glamorgan, Pontypridd, CF37 1DL, UK

Research highlights

- Lime content for optimal stabilisation of Lower Oxford Clay is about 6%.
- Lime-activated blastfurnace slag is a better binder for clay than lime or cement-activated slag.
- Blastfurnace slag requires only small quantities of lime for activation to stabilise clay.
- Lime-slag bound clay is durable enough for applications in road embankments in flood-prone areas.

Sustainable soil stabilisation for road embankments in flood plains

J. M. Kinuthia*, G. N. Obuzor, R. Robinson

Civil Engineering Research Unit, Department of Engineering, Faculty of Advanced Technology, University of Glamorgan, Pontypridd, CF37 1DL, UK

Abstract:

There is increasing need to meet the demand for road infrastructural development using sustainable technologies. This is more critical for road embankments in low-land areas, where high embankments consume large amounts of precious and scarce raw material resources. By applying and modifying locally available marginal and/or waste materials to improve their engineering properties, it is possible to achieve strong, durable and environmentally friendly construction materials, to a quality comparable or surpassing materials formulated using more expensive and increasingly unsustainable traditional stabilization materials such as Portland Cement (PC) and/or lime. This paper reports on a laboratory process that aimed at achieving this objective, using stabilised clay soil. Cylindrical test specimens were made using Lower Oxford Clay (LOC) as a possible target soil, stabilised with varying dosages of sustainable slag-based binders. The test specimens producing the highest compressive strengths were further investigated by immersion in water and resistance to expansion monitored for at least 28 days. The most successful formulations showed potential robustness upon prolonged flooding, suggesting viability. This is one example of possible approaches towards utilisation of scarce resources to achieve sustainable road infrastructural developments, especially in flood prone regions.

* Corresponding author, e-mail: jmkinuth@glam.ac.uk; Tel: +44(0)1443 482148; Mobile +44(0)7852756932

Key words — Flooding, stabilization, sustainability, Roads, embankment, soils.

1. Introduction

Road infrastructural development is critical for the development of all sectors of a nation's economy (Lee and Karunaratne, 2007). With the increasing environmental concerns there is need to employ innovative ways of strengthening low load-bearing capacity sub-grade soils for road pavements. This is particularly the case in the coastal regions and low lying areas such as flood plains, where the engineers face the problem of having to deal with poor soils of low load-bearing capacity, exacerbated by wave or flood-induced scour.

There are sophisticated approaches towards the understanding of the scour mechanisms (Huang et al., 2003; Myrhaug and Ong, 2009; Sobey, 2009; Young and Testik, 2009; among others). In road construction, the application of algorithms, numerical and prediction studies, modelling of coastal and estuarine flows, wetting and drying theories and finite element approaches is difficult and hard to simulate. To the credit of some of these studies, however, the mechanisms of wetting and drying fronts and the flow motions that create the vortices around structures of different properties that eventually induce scour are fairly well understood. It is therefore incumbent upon highway and materials engineers to formulate materials that best resist scour upon submergence, for road embankments in these low lying areas with predominantly poor soils.

According to Bell and Culshaw (2001), three main methods have been adopted to mitigate the effects of swelling and shrinkage of soils that may induce and exacerbate scour – use of foundations and structures that can tolerate movements without unacceptable damage;

48 isolation of the foundation and structure from the effects of the soil; and control of ground
49 conditions, particularly changes in moisture content and its effects. With the whole life
50 costing as a focal point of planning soil improvement works, soil stabilisation is one of the
51 common solutions (Ramana and Krishna, 2006; Nusier et al., 2007; Ayadat et al., 2008;
52 Ghosh et al., 2008).
53
54 Portland cement (PC) and/or lime have a traditionally been used for soil stabilisation, as an
55 alternative to aggregates, concrete or transporting large quantities of granular fills (Wild et
56 al., 1996). However, owing to recent global emphasis on reduced waste, natural resources
57 depletion, energy usage and low carbon dioxide (CO_2) emission – leading to reductions in
58 global warming, the use of natural and industrial waste and/or by-product materials as soil
59 stabilizers has become more common. This has the dual benefit of reduced cost and
60 increased sustainability (Higgins et al., 2002), and has motivated researchers in the search
61 for technological solutions to leap-frog sustainability and mitigating the effects of climate
62 change via low-carbon technologies.
63
64 There are many cutting edge technologies dealing with different levels of soil problems.
65 These range from complete replacement of low load-bearing soils with new aggregates,
66 chemical grouting and stabilisation with lime and/or Portland cement, or mechanical
67 stabilisation, to piling (O' Flaherty, 2002; Nusier et al., 2007 and Ayadat et al., 2008). More
68 sophisticated techniques such as vacuum preloading, dynamic consolidation methods and
69 the more recent explosive replacement techniques have also been deployed. The explosion
70 method for example, is an innovative technology that is reportedly being used in China (Chu
71 et al., 2006). It adopts a controlled explosion to create an excavation adjacent to a material

72 stockpile. The material then falls into the excavation and is then levelled off using typical
73 road construction plant. This, arguably, saves time by obviating the need for several passes
74 with construction plant to excavate and spread material as in normal road construction.
75
76 There is need to strike a balance between cost and environmental friendliness of the
77 construction process, and the ease of adoption of technologies. For this reason, this current
78 and on-going research aims at the development of an easily adoptable technology for
79 construction of road embankments in flood plains. The main aim is to stabilise low-bearing
80 capacity soils using sustainable means, and to a strength and durability level higher than
81 typical soil stabilisation using normal levels of lime (3-8%) or Portland cement (3-6%).
82 Cylindrical test specimens of 50mm in diameter and 100mm in length were made using
83 Lower Oxford Clay (LOC) stabilised with slag-based stabiliser blends of varying compositions.
84
85 The potential for damage upon flooding was estimated by monitoring linear expansion
86 characteristics of stabilised cylinder specimens as an indication of potential swell behaviour.
87 The expansions of the various stabilised soil mixtures were simultaneously monitored using
88 automatic linear expansion data logging apparatus until no further material changes were
89 evident. There are possibly other methods for assessing scour due to flooding, but all these
90 tests would only indicative of expected behaviour. For this reason, the assessment of
91 material expansion that typically precedes or accompanies disintegration of stabilised soil
92 material would also be a good tool to assess potential susceptibility to scour.
93

2. Materials

2.1. Soil

Lower Oxford Clay, (LOC) was used as the target soil for the investigation. The LOC was supplied by Hanson Brick Ltd., Stewartby, Bedford, UK. The mineralogical analysis carried out by Hanson Brick Ltd shows the LOC to contain 23% illite, 10% kaolinite, 7% chlorite, 10% calcite, 29% quartz, 2% gypsum, 4% pyrite, 8% feldspar and 7% organic substances (Higgins et al., 2002). Table 1 shows the oxide and mineralogical compositions, including other chemical composition data – carbonate, sulphate, sulphur (as sulphide), chloride and soluble silica. LOC was selected as the target soil in part because the thorough understanding of the soil by the researchers (Wild et al., 1996, 1998; Higgins et al., 1998, 2002; Oti et al., 2009), and partly because this soil is well-known for the problems encountered when it is stabilised in the traditional manner using lime (Jacqui, R.J., 1990a, 1990b; Snedker and Temporal, 1990; Snedker, 1996 among others). It was hypothesised that if the well-established and well-known expansive behaviour of this clay soil can be effectively managed, most of the problems presented by other soil types would likely be manageable to some reasonable degree. Ability to sustainably stabilise this soil with any proposed stabiliser would therefore be a good measure of success, and arguably, its use obviates the need to experiment with a wide variety of soils as LOC would typically represent a worst, or near worst, case scenario.

TABLE 1

2.2. Lime

116 The lime used was a quicklime (CaO), supplied by Buxton Lime Industries Ltd., in the form of
117 a white powder. Its oxide composition and some of its physical properties are presented in
118 Table 2.
119
120 *2.3. Portland Cement (PC).*
121 The Portland cement (PC) used for this study complied with BS: 1989/BS EN 197-1: 2000. It
122 was obtained from Aberthaw cement works, South Wales, United Kingdom, and its physical
123 and chemical composition is also shown in Table 2.
124
125 *2.4. Ground Granulated Blastfurnace Slag (GGBS)*
126 Ground Granulated Blastfurnace Slag (GGBS) is an industrial by-product material, resulting
127 from the manufacture of steel. The GGBS used in the current research work was supplied by
128 Civil and Marine Slag Cement Ltd, Llanwern, Newport, South Wales, United Kingdom. Table 2
129 shows its chemical composition and physical properties. One of the objectives of the current
130 research work was to utilise GGBS, a by-product material, as a key stabilising ingredient. It is
131 fairly commonly available in many countries and was therefore targeted for use in
132 conjunction with the traditional stabilisers of lime or Portland cement (PC) to stabilise clay
133 soils to a material with higher than the strength in a typical lime-stabilisation process with a
134 view to withstanding prolonged submergence in water.
135
136 TABLE 2
137
138 **3. Methodology**
139 The first stage of the current research work aimed at the stabilization of LOC using both the

140 traditional soil stabilisers of PC or lime, as controls, and also using more sustainable
141 stabilisers made by blending the traditional stabilisers with GGBS, a more sustainable
142 material being a by-product on which energy had already been expended. This phase of the
143 work used varying levels of lime, PC, lime - GGBS and PC - GGBS blends, as shown in Table 3.
144
145 For each stabiliser or stabiliser blend, nine cylindrical test specimens measuring 50mm in
146 diameter and 100mm in length were made from LOC mixed with the stabilizer at a pre-
147 determined moisture content of 23%. This optimal moisture content was determined from
148 compaction tests carried out in accordance to British Standard BS 1924-2 (1990) in order to
149 establish values of the maximum dry density (MDD) and optimum moisture (OMC) for the
150 LOC. A hand mixing technique was employed to enhance the homogeneity of the mix. The
151 wet material was subjected to a static compression in a steel mould, using a hydraulic jack,
152 so as to achieve the desired density. The cylinders were then extruded using a steel plunger.
153 The cylindrical specimens were then wrapped with cling film to ensure that further drying
154 was minimized.
155
156 At the end of each of the curing periods 7, 14 and 28 days, three cylindrical specimens per
157 mix composition were subjected to unconfined compressive strength (UCS) test at a strain
158 rate of 2mm per minute, using a Hounsfield Compression Testing machine, capable of
159 loading up to 10kN.
160
161 After carefully assessing the strength development of the stabilised LOC using the various
162 dosages of stabiliser and stabiliser blends, it was necessary to further fine-tune the mix
163 design to an optimal mix, with a view to further increasing strength. Additional test

specimens were made at different moisture content levels. Using the material formulations found to achieve maximum strength, more test specimens were made with a view to exploring their durability, by moist curing for 7 days and then subsequently soaking in water for a height of about 10mm. The soaking was maintained for a period of fifty days, during which linear expansion measurements were taken on a daily basis.

TABLE 3

4. Results

4.1. Unconfined compressive Strength

Figure 1 shows the unconfined compressive strength results when Lower Oxford Clay (LOC) was stabilised with increasing amounts of lime. With the exception of the unstabilised LOC (0% Lime), the strength increased with the age of curing. As the level of lime was increased, there was a clear optimum amount of lime between 4% and 8% lime content. Beyond this optimum of an average value of 6% lime, further addition of lime did not result in any further increase in strength. Thus, when 12% and 16% lime levels were used, the strength results were lower than those achieved with 4% and 8% lime dosages. No such optimum was observed when Portland Cement (PC) was used to stabilise the LOC (Figure 2), and the strength continued to increase with increasing stabiliser dosage, with the highest strength of slightly above 1000kN/m^2 being observed at 28 days of curing with 16% PC. Interestingly, at stabiliser dosage levels of 4% and 8% where lime showed optimal performance with strength values of about 700kN/m^2 at 28 days of curing, PC performed poorly relative to lime. While with lime the UCS values exceeded 600kN/m^2 beyond 14 days of curing, with PC, only after

187 curing for 28 days did the strength slightly go beyond this strength value when 8% PC was
188 used. Using 4% PC did not exceed 400kN/m^2 even after prolonged curing to 28 days. Lime
189 was therefore a more successful binder for the LOC used, relative to PC.
190
191 FIGURE 1
192
193 FIGURE 2
194
195 The results presented in Figure 3 show the strength values for different lime-GGBS stabiliser
196 blends of varying blending proportions, all at a total stabiliser dosage of 16% and at
197 compaction moisture content of 23%. The two mixes where unblended lime and unblended
198 GGBS were used may be regarded as controls. It is interesting to note from Figures 1 and 3
199 that using 16% lime did not appear to significantly influence the strength enhancement of
200 LOC with increase in curing time. As noted from Figure 1, increasing lime content appeared
201 to reach a level beyond which further addition of lime compromised strength development.
202 In contrast, from Figure 3, using 16% GGBS (without the presence of lime) showed a
203 significant increase in strength with increasing curing time. With the blended lime-GGBS
204 stabilisers, the strength appeared to mirror the observations observed with the separate
205 stabilisers, in that strength increased in inverse proportion with increase in lime, and in
206 direct proportion with the quantity of GGBS. Thus, as the lime content increased (and GGBS
207 content decreased) from 0%Lime-16%GGBS, the strength enhancement increased very
208 sharply such that maximum strength was achieved using 4%Lime - 12%GGBS. At 28 days of
209 moist curing, the maximum strength achieved was above 1200kN/m^2. Any further increase
210 in the amount of lime in the blended stabiliser resulted in reduced strength.

211

212 FIGURE 3

213

214 Results in Figure 4 show that when PC was used to blend with GGBS, the results were very
215 much similar to those observed with lime-GGBS blends, in the sense that the strength
216 enhancement rose sharply from 0%PC-16%GGBS to 4%PC-12%GGBS. However, at 28 days of
217 curing, the highest strength marginally fell short of the 1200kN/m^2 strength mark achieved
218 with the equivalent lime-GGBS blend. The results so far therefore show better performance
219 with lime or lime-GGBS relative to PC or PC-GGBS blended stabilisers. One notable difference
220 between the lime-system and the PC-system was when the stabilisers were used unblended
221 with GGBS. With lime, a dosage of 16% resulted in very low strength development while in
222 contrast, using 16%PC, showed relatively much higher strength values, only outperformed
223 by the 4%PC-12%GGBS optimal blend. This blended stabiliser achieved a marginally lower
224 strength value relative to its lime-GGBS equivalent, but within experimental error, the two
225 stabiliser blends achieved practically the same strength values at all the three curing periods
226 investigated. Lime used on its own achieved the lowest strength values.

227

228 In a flooding scenario, it is best to achieve the highest strength possible in any given system.
229 It was therefore found necessary to experiment with various parameter changes for the
230 lime- and PC-systems so far reported, with a view to exploring how further strength
231 development could be realised. For reason, it was considered necessary, for example, to
232 further explore the effect of changes in moisture content on the strength development
233 process. Driven by this, four levels of water contents were established by increasing the
234 moisture contents from 23%, at 5% increments to achieve data for four moisture contents

235 levels of 23, 28, 33 and 38%. From Figures 3 and 4, it was evident that the presence of GGBS
236 had a positive impact on both lime and PC in the stabilisation of LOC. As lime is the more
237 common traditional binder used for soil stabilisation, and having performed marginally
238 better than PC when blended with GGBS to stabilise LOC, the use of PC-GGBS blends was
239 therefore discontinued in further research work, in favour of lime-GGBS stabilisation. Other
240 considerations included cost and sustainability. In most parts of the world, lime is cheaper
241 than PC. In addition, the manufacture of PC is generally considered to have a more
242 significant negative environmental impact relative to that of lime, in terms of raw material
243 resources and energy inputs. The manufacture of both materials does contribute to
244 atmospheric carbon dioxide emissions, although at the volumes normally consumed, PC is
245 considered to be the greater contributor, further negating the advantages of its use as a soil
246 stabiliser compared to using lime.
247
248 FIGURE 4
249
250 The work carried out at the various compaction moisture contents was carried out using all
251 the mix compositions. This was found most logical, rather than pruning the range of mix
252 compositions based on the initial work carried out at a compaction moisture content of 23%
253 only. The results for the increased range of compaction moisture content are illustrated in
254 Figure 5. The results show that the stabiliser blends 4%Lime - 12%GGBS and 8%Lime -
255 8%GGBS continued to show the best performance at all the compaction moisture contents.
256 The results also show that from the initial strength values obtained at 23% moisture content,
257 further increase in moisture content improved strength development, up to 33% mc beyond
258 which further increase in mc lowered the strength values.

259

260 FIGURE 5

261

262 In a bid to improve the strength still further, it was thought that if the compaction effort
263 increased, higher strength values were achievable. Thus, further manipulation of the
264 compaction process was then carried out by increasing the weight of the test specimens
265 from 350g (density - 1.79 Mg/m^3) to 400g (density - 2.04 Mg/m^3). This work was only carried
266 out using the two optimal blends, and at three compaction moisture contents of 28, 33 and
267 38%. The results are shown in Figure 6. As speculated, there was a further increase in
268 strength for both the 4Lime-12GGBS and 8Lime-8GGBS blends at the various moisture
269 contents. The highest unconfined compressive strength value of about 2400 kN/m^2 was
270 recorded for the 4Lime-12GGBS at a moisture content of 28%.

271

272 FIGURE 6

273

274 *4.2. Linear Expansion*

275 In most civil engineering applications, the achievement of strength is only part of the
276 criteria. The more severe condition is the behaviour of the stabilised material upon wetting
277 due to exposure to water. For this reason, further investigations were carried out on the
278 durability of the stabilised mixtures upon soaking in water. This work was only carried out on
279 stabilised mixtures that had shown optimal strength development (4%Lime - 12%GGBS and
280 8%Lime - 8%GGBS). Linear expansion measurements were carried out on stabilised mixtures
281 compacted at the four compaction moisture contents of 23, 28, 33 and 38%. For control
282 experiments, identical investigations were carried out on mixtures stabilised with lime at the

283 same total stabiliser content (16%). The results are shown in Figure 7, and show that the
284 linear expansion of the control specimens stabilized with lime alone. Specimens compacted
285 at the lowest moisture content of 23% showed the highest expansion, reaching about 15%.
286 This is very much higher than the typical expansion of 4% recommend by the UK Department
287 of Transport for stabilised soil mixtures for road construction. The maximum linear
288 expansion decreased progressively as the compaction moisture content increased, thus, the
289 maximum compaction moisture content investigated of 38% shed the least expansion,
290 reaching a maximum of about 7%. Therefore, using lime as the sole stabiliser, all test
291 specimens failed the 4% linear expansion criteria.
292
293 FIGURE 7
294
295 Figure 8 shows the linear expansion test results for the test specimens stabilised using the
296 optimal lime-GGBS blended binder - 4Lime-12GGBS and 8Lime-8GGBS. The results at each
297 compaction moisture content are displayed together with their respective control using lime
298 alone. In all cases, the 4Lime-12GGBS stabiliser blend showed the lowest linear expansion
299 within the observation period of about 50 days. The other optimal stabiliser blend of 8Lime-
300 8GGBS also showed good expansion resistance compared to the control. For both optimal
301 blended stabilisers, there was a slight increase in expansion as the compaction moisture
302 content increased from 23% to 28%, although this decreased with further increase in
303 compaction moisture content.
304
305 For all the test specimens investigated, there was expansion during the 7-day moist curing
306 period prior to partial soaking in water. From past experience, a small amount of this

307 expansion, particularly that within the first day, is attributable to the relaxation of the test
308 specimens after extrusion from the steel mould. The more sustained expansion during the 7-
309 day period may be due to that taking place as a result of the early hydration of the stabilised
310 specimens. From Figure 8, a trend is evident, where there is increasing expansion upon
311 reduction in the compaction moisture content and upon increase in the quantity of lime in
312 the stabilised mixture.

313

314 FIGURE 8

315

316 **5. Discussion**

317 It is an established that changes in moisture content of a clay soil has a major effect on its
318 volume stability (Ramana Murty and Hare Krishna, 2006), causing costly damage to
319 pavement structures founded on clay soil (Lee and Karunaratne, 2007). It has also been
320 established that when lime is applied at a dosage within 1 – 3 wt%, it serves the general
321 purpose of soil modification, while any more addition above this level, depending on the clay
322 mineralogy and clay content, is normally added for the purpose of the more longer-term
323 stabilization (Wild et al., 1996). Research has shown that soil stabilization typically uses up
324 to 8 wt% lime or up to 5 wt% PC. However, for the interest of construction in flood-prone
325 areas such as flood plains, this paper investigated stabilisation at a higher stabiliser content,
326 using up to 16 wt% stabilizer. As this high stabiliser dosage comprises of only a low
327 proportion of lime, as lime is significantly replaced with the by-product material (GGBS), the
328 dosage used may therefore be regarded as less than double that typically used during
329 normal stabilisation with lime alone. Stabilization with lime improves the properties of clay
330 through flocculation of the individual clay particles into "floccs", which in turn adhere to in a

331 process commonly referred to as agglomeration. Both these processes form the early stages
332 of stabilisation, taking place within 72 hours of addition of lime and water to the clay. They
333 are accompanied and followed by the longer-term cementing effect, resulting from the
334 formation of cementitious products of chemical hydration reactions (mainly a complex
335 calcium silicate hydrate (C-S-H) gel (Wild et al. 1996)). When lime is introduced to a clay soil
336 followed in a stabilisation process, an exothermic reaction takes place resulting in drier soil
337 mixture, hence requiring higher moisture content for complete hydration reactions. Thus,
338 test specimens stabilised at 23% moisture content, the optimum moisture content value,
339 would require additional moisture in order to compensate for the drying effect of lime in
340 order to achieve optimal performance in terms of compressive strength.
341
342 Ground Granulated Blastfurnace Slag (GGBS), if present in the lime-clay mixture is activated
343 by the lime becoming part of the hydraulic binder system. In the current work the addition
344 of GGBS to the lime-stabilized LOC produced phenomenal results as shown by results from
345 both strength and linear expansion tests. This is believed to be as consequent of enhanced
346 C-S-H gel formation in the system, enhancing further blockage of the pores, thus also
347 reducing permeability (Wild et al., 1996) and improving strength. The slow reactivity sustains
348 the pozzolanic reactions for a relatively longer time compared to when lime is used on its
349 own as the stabiliser.
350
351 In addition to the extra strengthening effects attributed to the presence of GGBS due to its
352 activation by lime, previous research work by the authors (Wild et al., 1998; Oti et al. 2008)
353 and by Hughes and Glendinning (2004), has shown that sulphates, such as the gypsum
354 present in the LOC, can further promote GGBS activation and thus increase the chances of

355 the formation of more of the strength enhancing C-S-H gel. This would further reinforce the
356 consolidation of the matrix to give better strength properties to the system. Thus, it is
357 interesting to note that when 12% lime is substituted with GGBS to form the 4%-12%GGBS
358 blended binder, the maximum UCS value of about 2500kN/m^2 was obtained. Research has
359 also shown that laboratory-scale formulations of stabilised soil mixtures can be further
360 optimized in practice, with significant improvement in performance at a much reduced
361 stabiliser dosage and lower moisture content. This was demonstrated by Oti et al. (2009),
362 when the strength values of formulations for unfired stabilised soil bricks were more than
363 doubled, and stabiliser dosage more than halved, when materials volumes, mixing and
364 compaction procedures were carried out on an industrial-scale. Thus, the high strength
365 values, coupled with good water resistance of formulations in the current investigation
366 demonstrate the viability of the possible development of durable soil-based materials for
367 applications in low lying areas prone to sporadic submergence in water, in wet and flood
368 prone areas. For regions where iron and other metals are mined and processed, there is
369 scope for the development of a wide range of slag-based construction materials and
370 technologies. For example, despite the significant potential by GGBS as demonstrated in
371 the current work, its use in construction has predominantly been in its traditional application
372 in concrete such as in blast furnace (slag) cements.
373
374 The use of lime to stabilise a sulphate- or sulphide-bearing clay results in the formation of
375 ettringite – a complex product that engenders instability in the system by expansion
376 (Mitchell, 1986; Hunter, 1988; Snedker and Temporal, 1990; Snedker, 1996; Wild et al., 1996
377). In the presence of GGBS, no such excessive linear expansion was observed. According to
378 Higgins et al., (1998), clays stabilised with 6% lime were susceptible to swelling in the

379 presence of sulphate, while those stabilised with 1% lime + 5% GGBS were very resistant to
380 swelling. In the current work, both the 4Lime - 12GGBS and 8Lime - 8GGBS blend showed
381 good durability. The higher potential is likely to be in the 4Lime - 12GGBS binder, which has
382 good performance and also comprises of a higher proportion of the less expensive and more
383 sustainable GGBS, and less of the more expensive and environmentally unfriendly lime. For
384 the drier mixtures, the systems were observed to be unable to resist the water-induced
385 expansion upon soaking. This is perhaps due to incomplete hydration, as suggested by the
386 fact that when the hydration process is fully guaranteed by the presence of higher moisture,
387 the system produced rigidity to overcome the disruptive effect of soaking in water.

388

389 **6. Conclusions**

390 From the work reported in this paper, the following conclusions can be drawn, based on the
391 compressive strength and linear expansion results presented.

392 1. Lime-stabilized Lower Oxford Clay (LOC) attained maximum strength when about 6%
393 was used. Any further addition of lime did not show any further strength enhancement.
394 2. The LOC-Lime-GGBS system produced best strength results compared with LOC-Lime or
395 LOC-PC-GGBS. Maximum strength was achieved when the test specimens were
396 stabilized at a moisture content of 28%, which is wetter than the optimum moisture
397 content of the LOC of about 23%. The LOC-Lime-GGBS system also showed superior
398 linear expansion properties compared to the LOC-Lime system, with the systems
399 stabilised using 4Lime-12GGBS and 8Lime-8GGBS blended stabilisers showing best
400 performance.
401 3. The most successful formulation investigated – 4Lime-12GGBS – suggests possible
402 significant reduction in the current over-reliance on the traditional binders of lime

403 and/or Portland cement to stabilise soils. The combined qualities of high strength,
404 minimal expansion and the economy associated with the incorporation of waste and/or
405 by-product materials suggests commercial viability and a prolonged supply of currently
406 dwindling natural raw material resources. Combined technological, environmental and
407 economic advantages in the formulations investigated strengthen sustainability,
408 especially in terms of applications in road construction in low-lying areas where good
409 strength development coupled with enhanced water resistance upon flooding is
410 essential for the maintenance of a robust road network.
411 4. Further research should be carried out to evaluate other performance and sustainability
412 criteria such as loss of strength upon soaking, pore structure and permeability, together
413 with economic and environmental assessments. These are the subject of on-going work
414 and will be reported in future publications.
415

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424 Ground Granulated Blastfurnace Slag and Portland cement respectively.
425

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493 semicircular breakwaters. Coastal Engineering. 56, 868-875.
494
495

LIST OF TABLES AND FIGURES

520 Figure 5 - Unconfined Compressive Strength (UCS) vs. curing period for LOC stabilised using
521 various Lime-GGBS blended binder of varying proportions, at (a)-28%, (b)-33%, and (c)-38%
522 moisture content, and at a compaction density of 1.79 Mg/m^3.
523
524 Figure 6 - Unconfined Compressive Strength (UCS) vs. curing period for LOC stabilised using
525 the two optimal Lime-GGBS blended binders (a) 4L-12GGBS and (b) 8Lime-8GGBS, moisture
526 contents of 28, 33%, and 38%, and at the increased compaction density of 2.04 Mg/m^3.
527
528 Figure 7 – Linear expansion of LOC stabilised using 16% Lime for cylinder specimens
529 compacted at moisture contents of 28, 33%, and 38%, during moist curing for 7 days
530 followed by soaking in water up a total observation period of 50 days.
531
532 Figure 8 – Linear expansion of LOC stabilised using 16% stabiliser for cylinder specimens
533 compacted at moisture contents of 28, 33%, and 38%, during moist curing for 7 days
534 followed by soaking in water up to a total observation period of 50 days.
535

Table 1: Oxide, mineral and chemical composition analyses of the Lower Oxford Clay (LOC) (Higgins et al, 2002; Wild et al, 1998).

Oxide Analysis	**Composition (wt%)**
SiO_2	46.73
Al_2O_3	18.51
CaO	6.15
Fe_2O_3	6.21
MgO	1.13
K_2O	4.06
Na_2O_3	0.52
TiO_2	1.13
FeO	0.80
Mn_2O	0.07
P_2O_5	0.17
L.O.I	15.79
Mineral Analysis	
Chlorite	7
Illite	23
Gypsum	2
Kaolinite	10
Quartz	29
k-feldspar	8
Calcite	10
Pyrite	4
Chemical and other Analyses	
Soluble Silica	0.43
Carbonate (CO_3)	5.02
Chloride (Cl)	0.01
Sulphide (S_2)	0.018
Sulphate (SO_4)	1.29
Total Sulphur	1.50
Organics	7

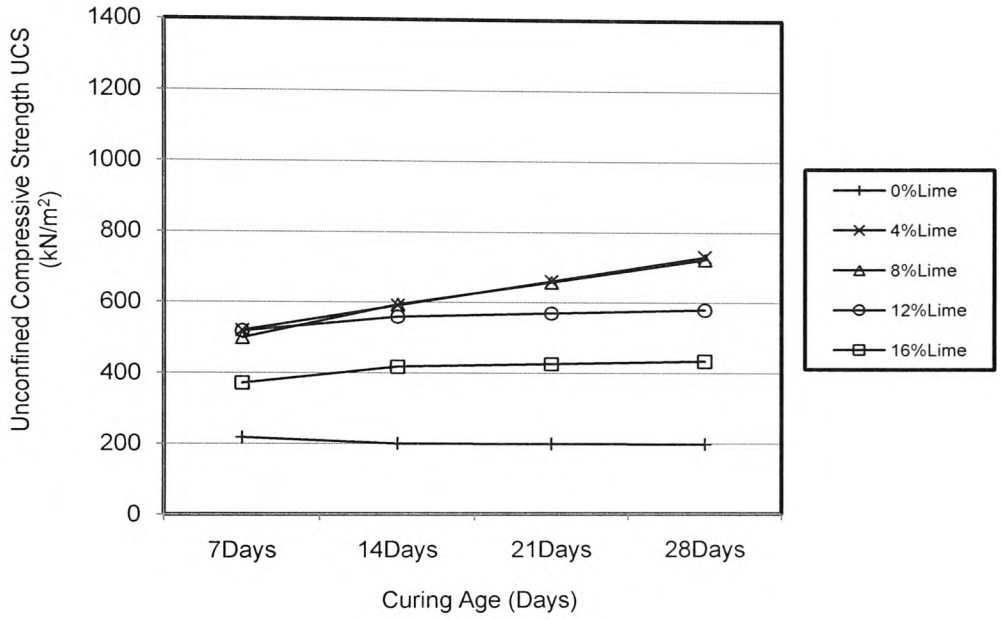
Table 2: Oxide and chemical composition, and some physical properties of quicklime, Portland cement and GGBS.

Oxide Analysis	**Composition (wt%)**		
	Quicklime	**Portland Cement**	**GGBS**
SiO_2	0.9	20	35.34
TiO_2	-	-	-
Al_2O_3	0.15	6	11.59
Fe_2O_3	0.07	3	0.35
MnO	-	0.03-1.11	0.45
$CaCO_3$	2.2	-	-
CaO	95.9	63	41.99
MgO	0.46	4.21	8.04
S_2	-	-	1.18
SO_3	-	2.3	0.32
Physical properties			
Specific gravity	2.3	3.15	2.9
Bulk Density, kg/m^3	480	1400	-
Colour	White	Grey	Off-white
Insoluble Residue	-	0.5	0.3
Glass Content	-	-	≈ 90

Table 3 – Mix compositions used in the study

Target material	Stabilisers ingredients and dosage (%)		Stabiliser blending ratio
	Lime or PC	GGBS	Blending Ratio
Lower Oxford Clay (LOC)	0	16*	0 : 16
	4	12	1:3
	8	8	1 : 1
	12	4	3 : 1
	16*	0	16 : 0

* - Control mixes


Unconfined Compressive Strength UCS (kN/m2)
1400
1200
1000
800
600
400
200
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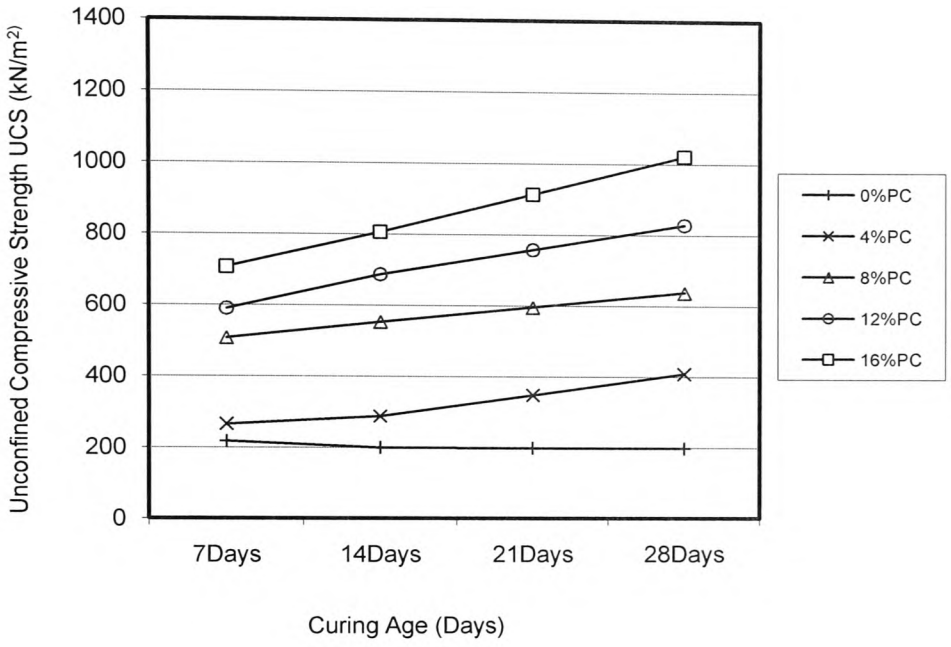
7Days
14Days
21Days
28Days
Curing Age (Days)
0%Lime
4%Lime
8%Lime
12%Lime
16%Lime

Figure 1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200
0
Unconfined Compressive Strength UCS (kN/m2)
7Days
14Days
21Days
28Days
Curing Age (Days)
0%PC
4%PC
8%PC
12%PC
16%PC

Figure 2

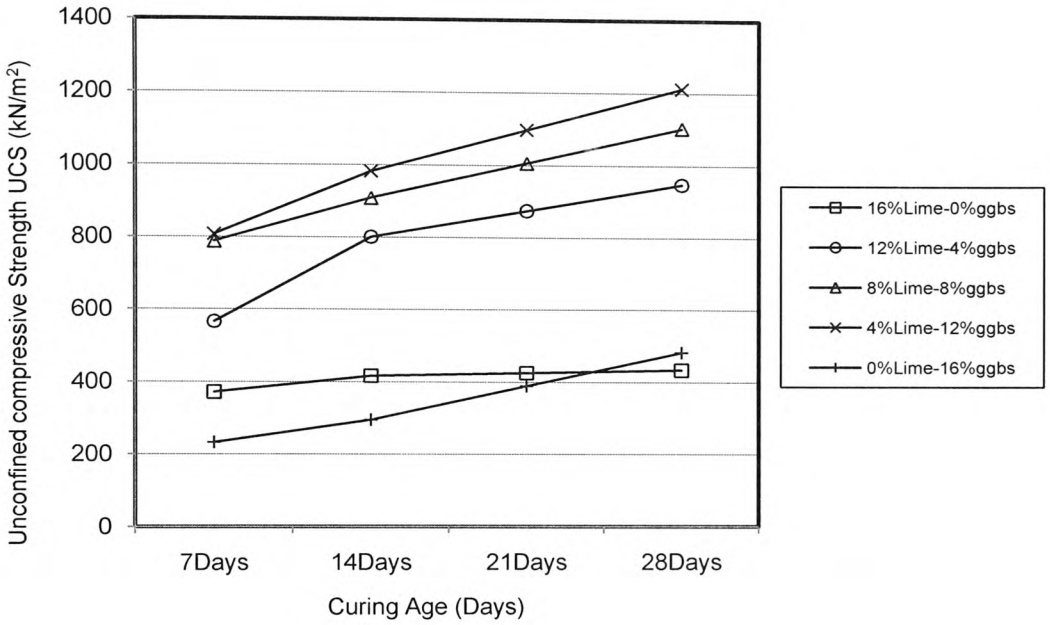

1400
1200
1000
800
600
400
200
0
Unconfined compressive Strength UCS (kN/m²)
7Days
14Days
21Days
28Days
Curing Age (Days)
16%Lime-0%ggbs
12%Lime-4%ggbs
8%Lime-8%ggbs
4%Lime-12%ggbs
0%Lime-16%ggbs

Figure 3

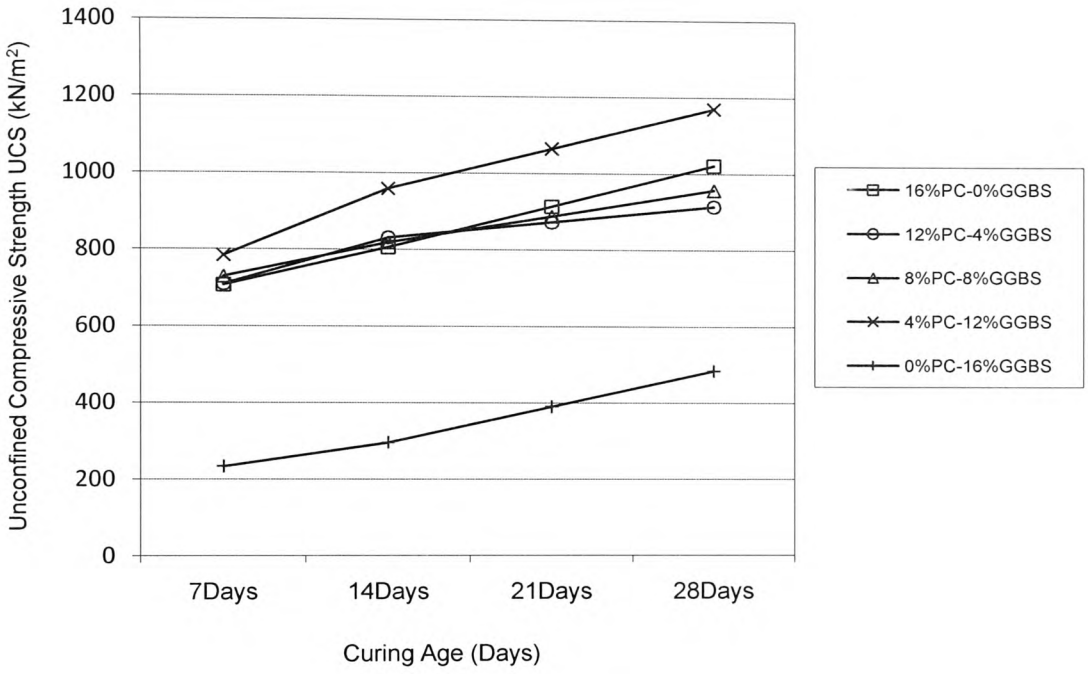
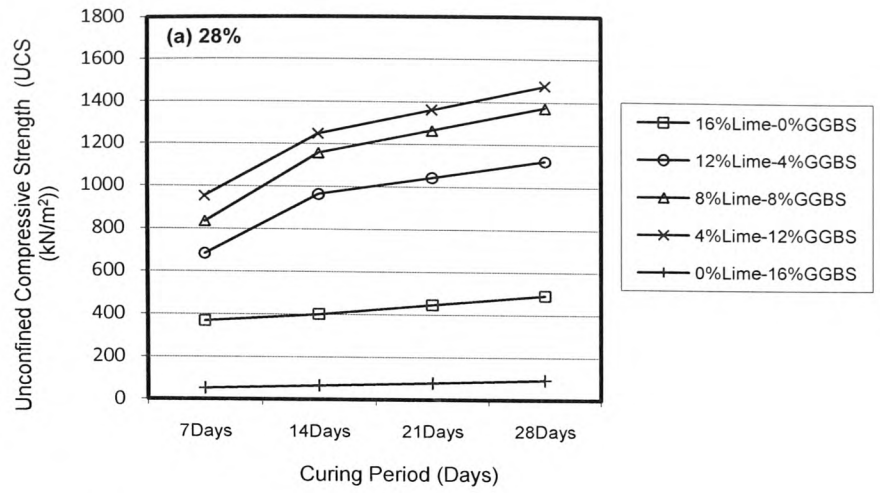
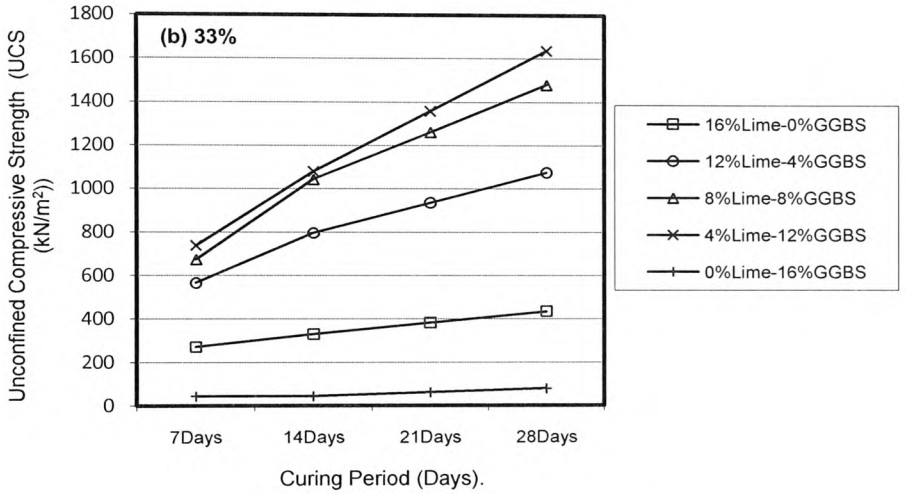

1400
1200
1000
800
600
400
200
0
Unconfined Compressive Strength UCS (kN/m²)
7Days
14Days
21Days
28Days
Curing Age (Days)
16%PC-0%GGBS
12%PC-4%GGBS
8%PC-8%GGBS
4%PC-12%GGBS
0%PC-16%GGBS

Figure 4


(a) 28%
Unconfined Compressive Strength (UCS (kN/m²))
1800
1600
1400
1200
1000
800
600
400
200
0
7Days
14Days
21Days
28Days
Curing Period (Days)
16%Lime-0%GGBS
12%Lime-4%GGBS
8%Lime-8%GGBS
4%Lime-12%GGBS
0%Lime-16%GGBS


(b) 33%
Unconfined Compressive Strength (UCS (kN/m²))
1800
1600
1400
1200
1000
800
600
400
200
0
7Days
14Days
21Days
28Days
Curing Period (Days).
16%Lime-0%GGBS
12%Lime-4%GGBS
8%Lime-8%GGBS
4%Lime-12%GGBS
0%Lime-16%GGBS

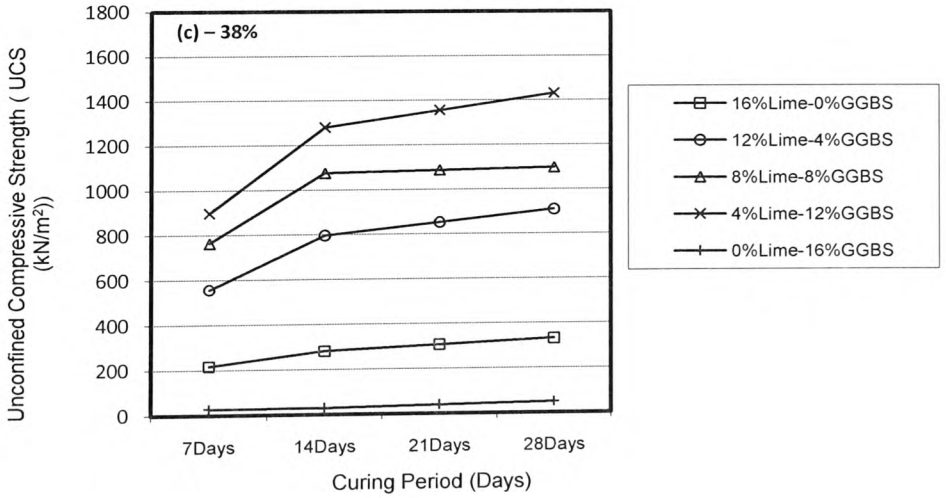
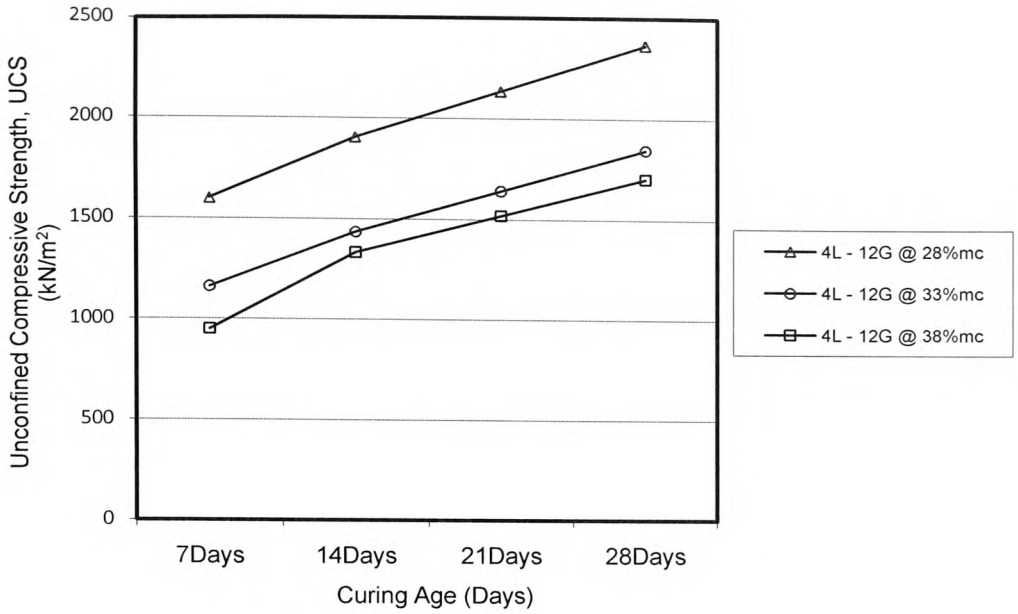

(c) – 38%
Unconfined Compressive Strength (UCS (kN/m²))
1800
1600
1400
1200
1000
800
600
400
200
0
7Days
14Days
21Days
28Days
Curing Period (Days)
16%Lime-0%GGBS
12%Lime-4%GGBS
8%Lime-8%GGBS
4%Lime-12%GGBS
0%Lime-16%GGBS

Figure 5


2500
2000
1500
1000
500
0
Unconfined Compressive Strength, UCS (kN/m²)
7Days
14Days
21Days
28Days
Curing Age (Days)
4L - 12G @ 28%mc
4L - 12G @ 33%mc
4L - 12G @ 38%mc

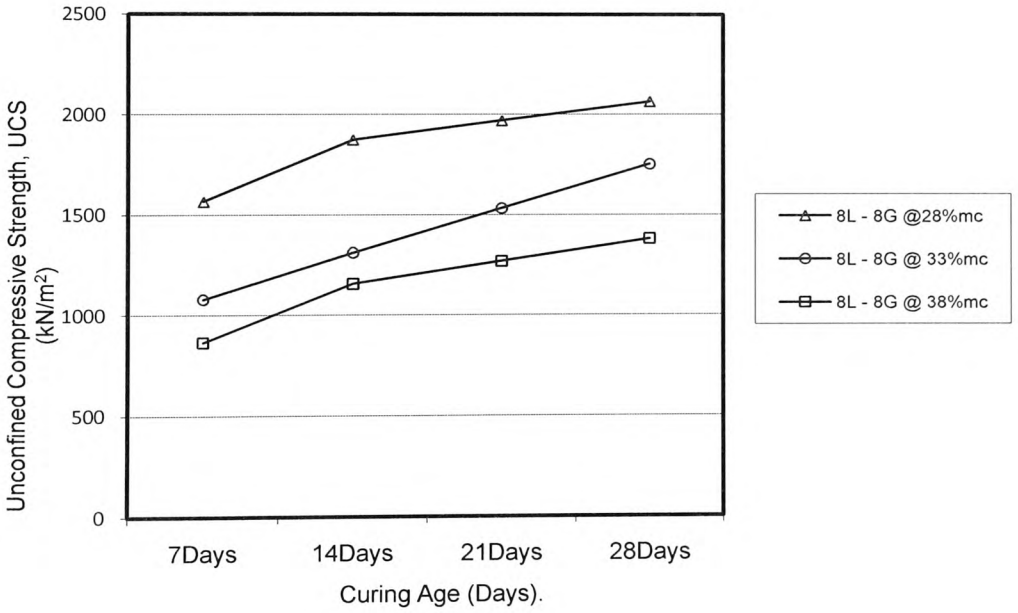

2500
2000
1500
1000
500
0
Unconfined Compressive Strength, UCS (kN/m²)
7Days
14Days
21Days
28Days
Curing Age (Days).
8L - 8G @28%mc
8L - 8G @ 33%mc
8L - 8G @ 38%mc

Figure 6

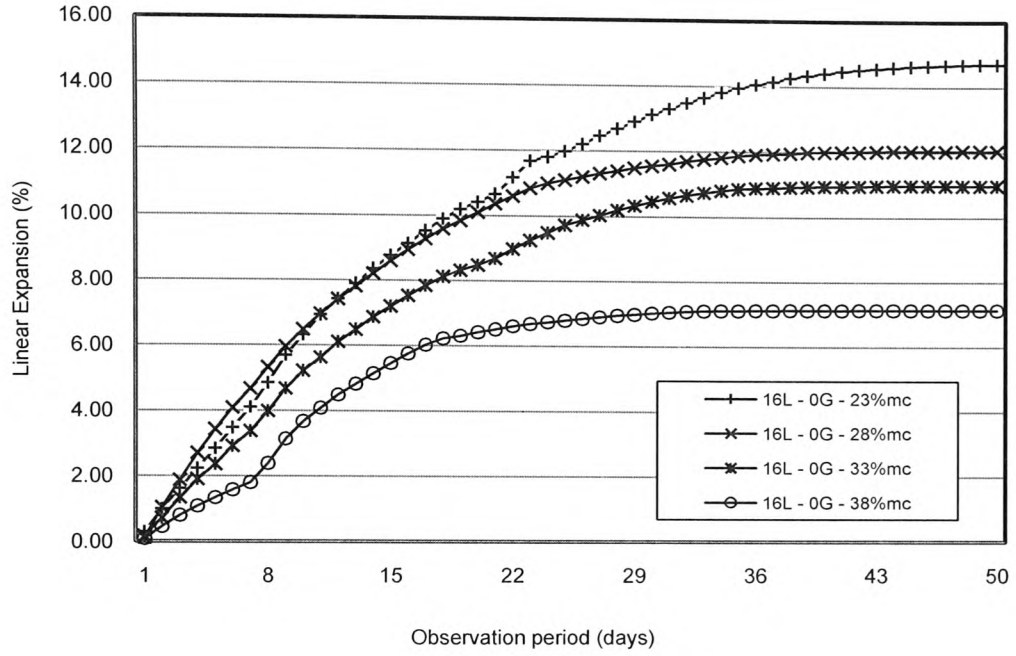

16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
Linear Expansion (%)
1
8
15
22
29
36
43
50
Observation period (days)
16L - 0G - 23%mc
16L - 0G - 28%mc
16L - 0G - 33%mc
16L - 0G - 38%mc

Figure 7

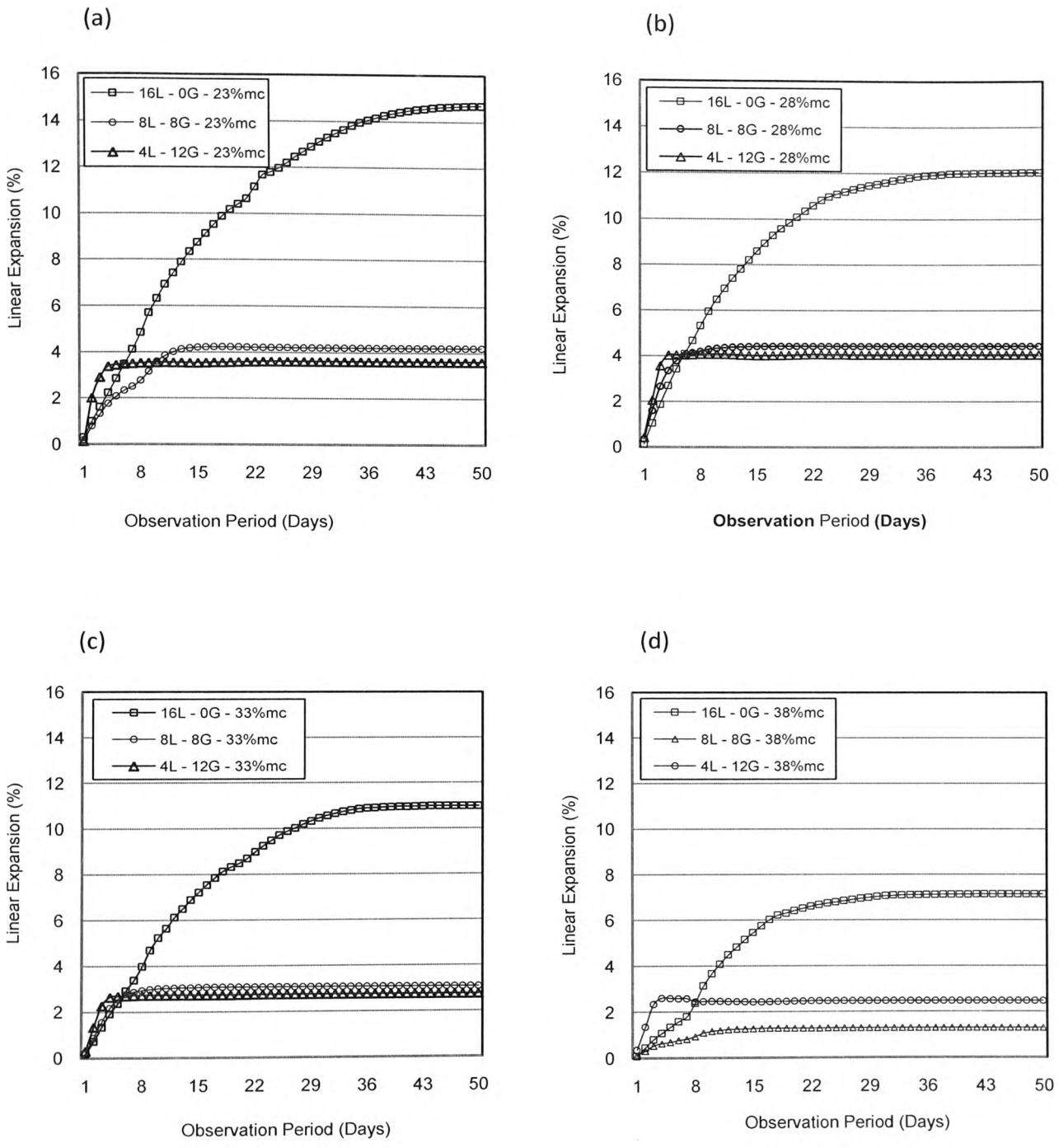

(a)
16L - 0G - 23%mc
8L - 8G - 23%mc
4L - 12G - 23%mc
Linear Expansion (%)
Observation Period (Days)
(b)
16L - 0G - 28%mc
8L - 8G - 28%mc
4L - 12G - 28%mc
Linear Expansion (%)
Observation Period (Days)
(c)
16L - 0G - 33%mc
8L - 8G - 33%mc
4L - 12G - 33%mc
Linear Expansion (%)
Observation Period (Days)
(d)
16L - 0G - 38%mc
8L - 8G - 38%mc
4L - 12G - 38%mc
Linear Expansion (%)
Observation Period (Days)

Figure 8